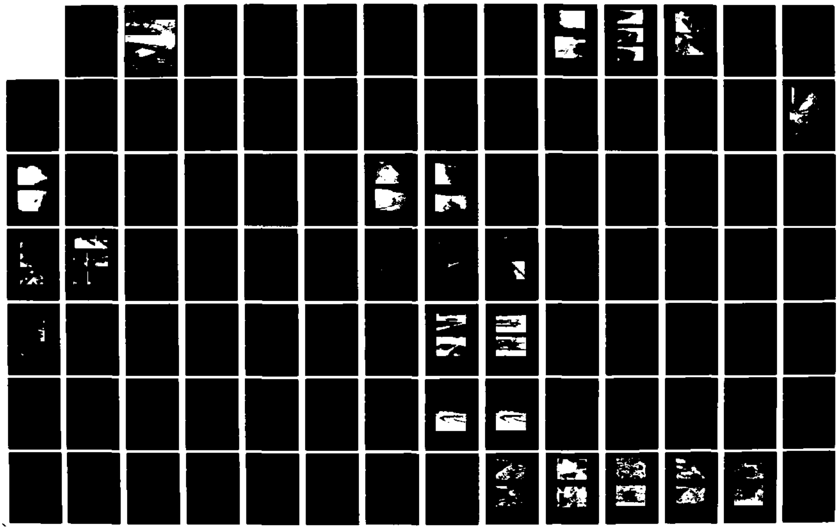
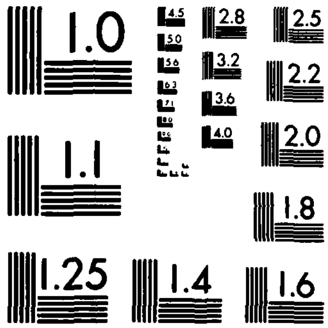
AD-A121 140 THE STREAMBANK EROSION CONTROL EVALUATION AND DEMONSTRATION ACT OF 1974 S..(U) ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG MS HYDRA.. 1/3

UNCLASSIFIED M P KEOWN ET AL. DEC 81 F/G 13/2 NL





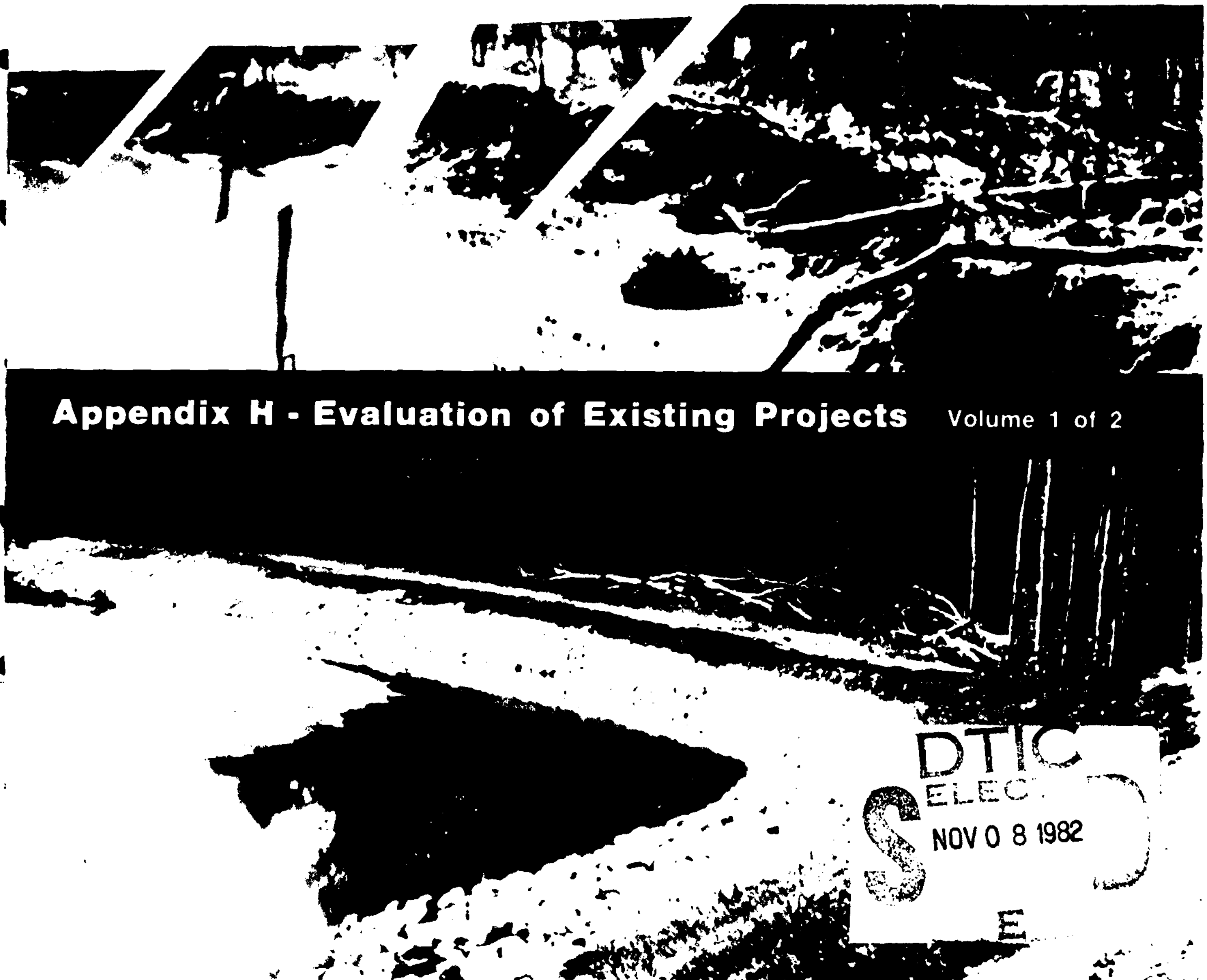
MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

(12)

US Army Corps of Engineers

December 1981

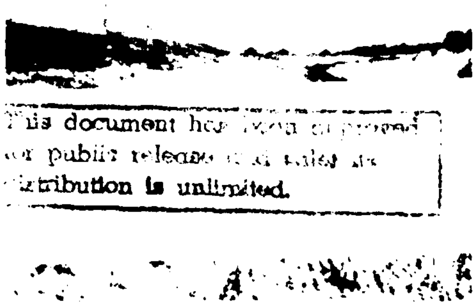
THE STREAMBANK EROSION CONTROL
EVALUATION AND DEMONSTRATION ACT OF 1974
SECTION 32, PUBLIC LAW 93-251



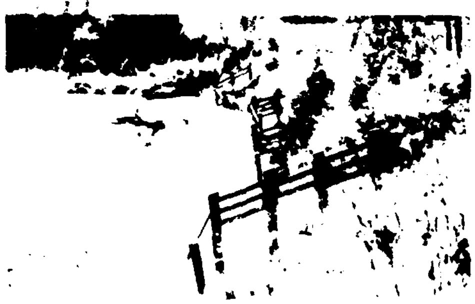
Appendix H - Evaluation of Existing Projects

Volume 1 of 2

Rock Toe With Tie-Backs



Precast Block Paving



Board Fence Dikes

FINAL REPORT TO CONGRESS

THE STREAMBANK EROSION CONTROL
EVALUATION AND DEMONSTRATION ACT OF 1974
SECTION 32, PUBLIC LAW 93-251

APPENDIX H
EVALUATION OF EXISTING PROJECTS

VOLUME 1 OF 2

Consisting of

A BRIEF SUMMARY REPORT AND INDIVIDUAL EVALUATION REPORTS ON FIFTY EXISTING STREAMBANK EROSION CONTROL PROJECTS CONSTRUCTED PRIOR TO OR SEPARATE FROM THE SECTION 32 PROGRAM

U.S. ARMY CORPS OF ENGINEERS
December 1981

APPENDIX H

EVALUATION OF EXISTING PROJECTS

CONTENTS

VOLUME 1 of 2

CONTENTS (Continued)

VOLUME 2 of 2

PREFACE

The evaluation of existing projects reported in this appendix was authorized by the United States Congress in Section 32 of the Water Resources Development Act of 1974 (Public Law 93-251), as amended. The work was accomplished as a combined effort between engineers of the U. S. Army Engineer Waterways Experiment Station (WES) and engineers from the various U. S. Corps of Engineers Districts where the existing projects are located. These evaluations were prepared by numerous individuals at WES and in the various Corps of Engineers Districts.

Special acknowledgment is given to Messrs. Malcolm P. Keown and Elba A. Dardeau, Jr., and Mrs. Etta M. Causey of the Environmental Laboratory at WES and Dr. Edward B. Perry of the Geotechnical Laboratory at WES, Mr. Walter Linder of MRK, all participating Corps of Engineers Districts and Divisions, and cooperative public and private landowners for their assistance in collecting, compiling, and writing much of the following text.

TECHNICAL SUMMARY

A variety of existing streambank works (built before the Streambank Erosion Control Evaluation and Demonstration Act of 1974) at 50 projects throughout the United States were selected for limited observation, monitoring, and evaluation using previous field observations and data and information acquired during the Section 32 Program. These existing projects were chosen to represent a wide variety of streams, soils, and bank protection techniques. The evaluation of these existing projects allowed determination of performance for various protection methods. These findings supplement the evaluations of the other projects constructed under the Section 32 Program. See Exhibit 1 for locations of the existing projects and Exhibits 2, 3, and 4 for typical projects evaluated. A detailed report on each of these projects is provided in this appendix; a summary of information on these projects is given in Exhibit 5.

Channel Characteristics and Erosion Problems

Summary and Range of Streambank (Geotechnical) and Flow (Hydraulic) Characteristics

The streambanks and beds of the 50 existing projects vary from homogeneous clays, sands, silty sands, or gravels to heterogeneous banks of numerous soil compositions. Bank slopes varied from near vertical to 1V on 5H with bank heights ranging from about 4 to 40 ft. Groundwater levels, channel bed gradients, and streamflows are generally representative of most small to medium streams in the United States. Discharges and velocities range from 0 to 865,000 cfs and 0 to 12 fps, respectively.

Causes of Erosion and Failures

The major causes of bank erosion that required design and construction of the existing projects were:

- Channel bed degradation
- Streamflow
- Water-level fluctuations
- Wave action

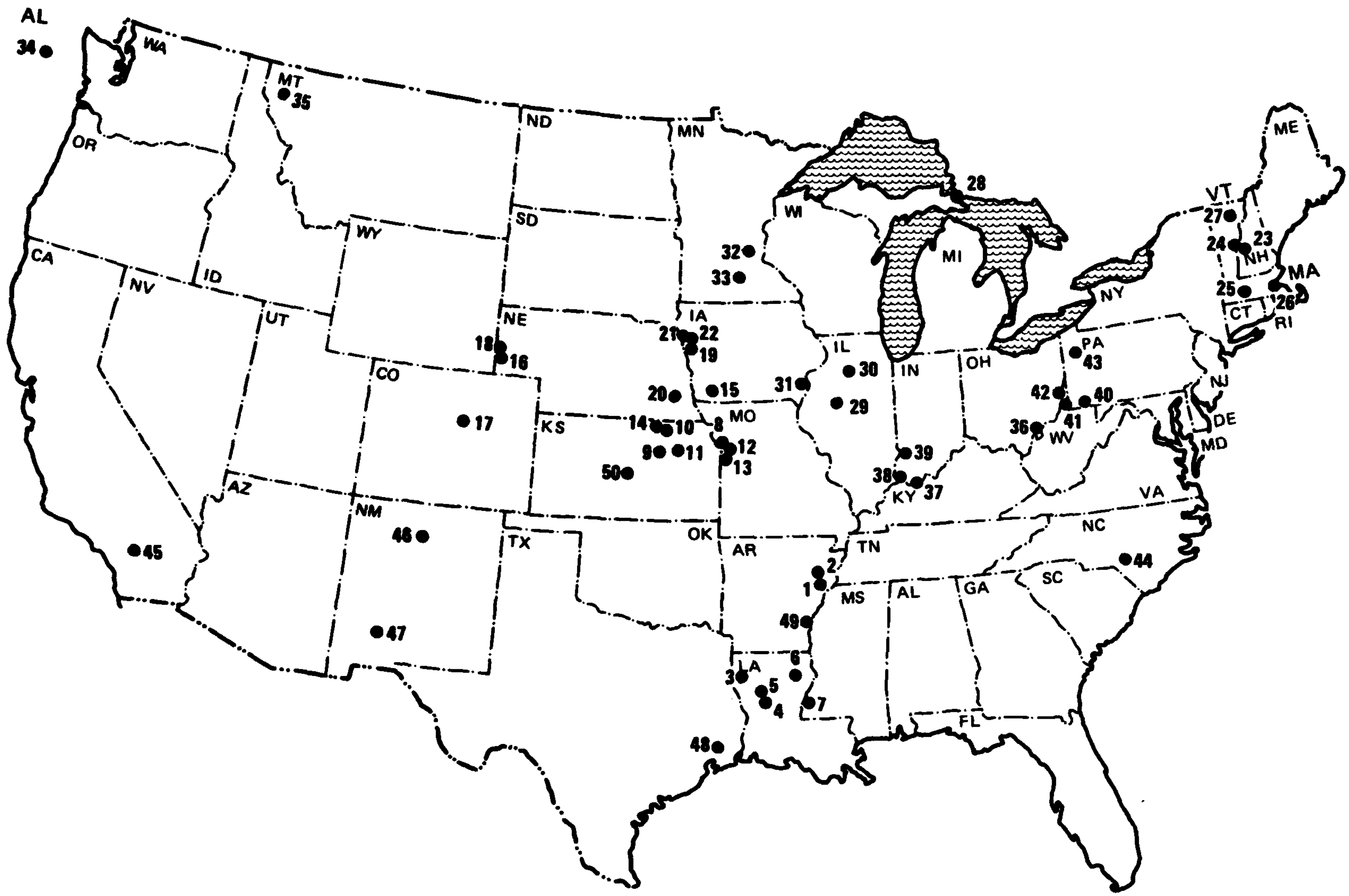


Exhibit 1. Locations of existing projects

Connecticut River at Thetford, Vermont

Monogahela River (left bank) near California, Pennsylvania

Exhibit 2. Stone-filled tire revetment constructed
by property owners (successful)

5 May 1979, discharge is 55,000 cfs, slumping and erosion of riverbank on downstream end, ice revetment still in place on upstream end

12 May 1979, river discharge is 27,400 cfs, ice revetment completely melted, riverbank erosion underway on upstream end

4 June 1979, river discharge is 30,200 cfs, riverbank still frozen, erosion progressing rapidly

Exhibit 3. Ice revetment, Tanana River, Alaska (failure)

Typical riprap revetment, constructed by the Corps of Engineers (successful)

Exhibit 4. Ohio River (left bank) at Cloverport, Kentucky. Downstream view shows toe of revetment under wave attack due to passing barge traffic (19 June 1978)

High-stage streamflow in the various channel alignments and river stages were considered to be the most frequent cause of subsequent erosion and failure observed at nine of the existing projects that have experienced any damage. Six of these nine projects were flanked during high-stage streamflow. Channel bed degradation was the most significant failure mechanism necessitating these 50 projects as shown in Exhibit 5. Multiple causes were identified at many projects including a combination of the four causes listed above and other less frequent causes such as overbank flow, seepage, ice/debris, and freeze/thaw.

Types of Protection and Relative Costs at Existing Projects

General Description

A general physical description of the protection methods used for the 50 existing projects is given in Exhibit 5.

Relative Costs

Cost data for most of the existing projects were not comparable due to the variations occurring over the period during which the projects were constructed (1939-1977).

Monitoring and Observations of Existing Projects

Monitoring of the existing projects after collection of available data consisted of field inspections and evaluations. Many of the existing projects were of advantage to the program because they had experienced several flood flows. Historical discharge data and past performance were collected for each existing project.

Maintenance and Rehabilitation of Existing Projects

Rehabilitation or maintenance of existing projects was not required under the Section 32 Program.

Summary of Findings

A wide range of geologic and hydraulic conditions, erosion, failure mechanisms, and protection techniques are represented by the 50 existing sites located throughout the Nation. The evaluation of these existing sites added significantly to the overall Section 32 Program evaluation because of the variety of conditions and the longer time period that the existing sites had experienced flow.

Significant Observations

- The Winooski River project, Vermont, constructed by the Soil Conservation Service (SCS) in the late 1930's at the request of local landowners and monitored last in 1980 by the New York District, Corps of Engineers, is perhaps the most unique of the 50 projects because of the length of time since project construction (40 yr) and the general success of erosion control. The two sites observed, where the temporary stone-filled log cribbing and hand-placed riprap structures were constructed, indicated no sign of erosion on the streambanks with various types of vegetation providing good bank coverage above the normal water surface. Additional details and findings are contained in this appendix.
- Used tires filled with stone were used successfully by private residents at four existing projects (two shown in Exhibit 2). These projects were highly cost-effective due to landowners collecting free materials and doing the work themselves.
- Rock and sheet-pile grade-control structures were effective in the prevention of channel bed degradation. Gabions were also used at one project for grade control.
- Vegetation has been successfully used on upper banks in conjunction with structural protection on the lower bank.
- Gabions were effective in establishing a low-flow channel with vegetated upper banks.
- Manufactured alternatives of precast concrete slabs and blocks for

bank protection had a higher failure rate than the most conventional bank protection.

- Permeable spur dikes constructed of board fencing anchored to steel piling have generally been unsuccessful at two existing projects.
- Soil cement was used to form riprap at one existing project. Site specific testing of this procedure is needed to determine application and curing rates.
- Kellner jacks were successfully used at three existing projects. Proper installation (alignment, anchoring, and spacing) is required and some future maintenance should be anticipated.
- Wire fence retards were used successfully on several existing projects. The fence promotes sediment deposition and growth of vegetation along the channel side slopes. Proper fencing design requires toe protection to prevent undermining and has proved cost-effective on many small streams.
- Ice revetment was a new concept tried on the Tanana River in Alaska with unsuccessful results (Exhibit 3).
- Rock is the most commonly used material for protection against streambank erosion, although the methods of placement and design vary widely.

Conclusions

- The most cost-effective means observed for protecting streambanks against erosion was where landowners on smaller rivers used locally available materials (used tires filled with stone) and did the work themselves.
- Thirty-two of the fifty existing projects incorporate some type of stone ranging from total riprap revetment (e.g., quarry-run or graded-stone blankets; see Exhibit 4) to trench-fill longitudinal toe protection, grade-control structures, stone-filled fences, stone-covered lumber mats, timber cribs filled with stone, used tires filled with stone, pile dikes with stone fill, gabion mats,

and stone dikes. All of these methods have provided protection ranging from poor to excellent, with the majority being rated as good. The project rated fair required repairs due to partial failure.

- Riprap revetment was the first choice of bank protection where stones of sufficient size were available because of durability and other advantages.
- A riprap revetment is flexible and is neither impaired nor weakened by slight movement of the bank resulting from settlement or other minor adjustments.
- Local damage or loss is easily repaired by the placement of more rock.
- Construction using rock is not usually complicated and no special equipment or construction practices are necessary.
- Riprap is recoverable and may be stockpiled for future use.
- The cost-effectiveness of quarry-run stone for long-term protection in comparison with other protection types is usually a very effective protection.

Significant Participation by Other Organizations

The 50 existing projects were constructed by the Soil Conservation Service, private interests, and U. S. Army Corps of Engineers Districts as indicated in Exhibit 5. Design, construction, and performance data for the existing projects were obtained from these agencies. Historical flow data for many of the projects were obtained from the U. S. Geological Survey.

Exhibit -5

SUMMARY OF EXISTING BANK STABILIZATION PROJECTS

Map No.*	Stream Project Location CE Office Year Completed	Erosion Agent	Protection Method	Present Condition and Remarks
1	St. Francis River Clarks Corner, AR Memphis 1964	Eddy currents set up around bridge pier	Stone riprap on lumber mattress (lower bank) and riprap on filter fabric (upper bank)	Excellent. Bridge abutment endangered by scour pocket which cut into roadway on downstream side of bridge
2	Caney Creek Caney Creek, AR Memphis 1975	Streamflow over highly erodible bank soils	Lime and gypsum treatment, clay gravel lining, vegetation	Excellent. Test channel in dispersive clay; project constructed by SCS
3	Red River Morameal, LA New Orleans 1975	High stage streamflow against concave bank of bendway	Local and specified stone, sand-filled bags, soil-cement blocks, gabions, and cellular block on upper bank	Very good. Protect levee and reduce bank erosion. Only high water in April 1979
4	Red River Fausse, LA New Orleans 1975	High stage streamflow against concave bank of bendway	Trench-fill and pile revetment, pile dikes w/stone fill	Excellent. Reduce bank erosion and maintain channel alignment
5	Red River Perot, LA New Orleans 1970	High stage streamflow against concave bank of bendway	Permeable timber fence dikes	Upstream end of dike field lost. Protects pipeline crossing; 5-year design life; major repair and upstream extension required in 1978
6	Big Creek Big Creek, LA Vicksburg 1977	Channel realignment resulted in a steeper bed gradient and higher flow velocities; grade control was necessary to prevent bed degradation and bank failure	Sheet-pile weir structures with stone riprap upstream and downstream of pilings	Good. Part of channel enlargement project
7	St. Catherine Creek Natchez, MS Vicksburg 1973	High stage streamflow against concave bank of bendway	Local materials, tires, and timber piles	Good. Bank protection constructed by local resident
8	Little Blue River Independence, MO Kansas City 1978	Streamflow over highly erodible bank soils	Riprap on side slopes of low-flow channel with short horizontal blanket at toe	Excellent. Protects side slopes of low-flow channel
9	Republican River Milford Dam, KS Kansas City 1969	Streamflow over highly erodible soils	Stone riprap revetment with horizontal toe blankets; four test sections, with various toe configurations	Very good. Site located on outlet channel of Milford Dam
10	Little Timber Creek Frankfort, KS Kansas City 1963	Channel realignment resulted in a steeper bed gradient and higher flow velocities; grade control was necessary to prevent bed degradation and bank failure	Series of sheet piling and rock sills	Good. Structures prevent channel degradation and subsequent damage to adjacent levees

(Continued)

* See Exhibit VIII-1 for project locations.

Map No.	Stream Project Location CE Office Year Completed	Erosion Agent	Protection Method	Present Condition and Remarks
11	Mud Creek Lawrence, KS Kansas City 1978	Channel realignment would result in a steeper bed gradient and higher flow velocities; grade control was necessary to prevent bed degradation and bank failure	Four sheet piling and rock sills	Excellent. Structures prevent channel degradation
12	Little Blue River Independence, MO Kansas City 1978	Channel realignment would result in a steeper bed gradient and higher flow velocities; grade control was necessary to prevent bed degradation and bank failure	Sheet piling and rock sills in low-flow channel	Excellent. Structures prevent degradation of low-flow channel
13	Little Blue River Independence, MO Kansas City 1978	Streamflow over highly erodible soil	Noncohesive materials replaced by seeded clay blanket	Good. Protects high-flow channel berm and side slopes
14	Big Blue River Near Marysville, KS Kansas City 1977	High stage streamflow against concave bank of bendway	Fencing with rock-dike tiebacks	Severe damage to fencing. Structure placed to protect county road and right abutment of bridge
15	102 River Bedford, IA Kansas City 1974	High stage streamflow through relatively straight reaches and channel bed degradation	Fabriform mat	Failed. Protection of bridge abutment, dam abutment, and bank. Undercutting of mat led to failures
16	Gering Drain Near Gering, NE Omaha 1969	Streamflow resulting in channel degradation and flow over highly erodible bank soil	Double-row fencing filled with stone or hay bales	Very good. Fencing is part of plan to prevent rapid enlargement or drains
17	Plum Creek Near Denver, CO Omaha 1970	High stage streamflow against concave bank of bendway	Woven wire fencing on steel rail post, stone root, and four perpendicular stone dikes	Excellent. Protects waterline crossing
18	Gering Drain Gering, NE Omaha 1969	See erosion agent under Site 16	Several low broad-crested rock sills	Very good. Sills are part of plan to prevent rapid enlargement of drains
19	Little Sioux River Onawa, IA Omaha 1969	Overbank flow	Gabion mattresses	Fair. Protection of stilling basin side slopes when high flows bypass drop structure and reenter channel as overbank flow
20	Deadman's Run and Antelope Creek Lincoln, NE Omaha 1979	Channel realignment resulted in a steeper bed gradient and higher flow velocities; grade control was necessary to prevent bed degradation and bank failure	Gabion baskets along base of side slopes with grass seeding on upper bank; gabion drop structures	Excellent. Channel was realigned to accommodate urban development

(Continued)

Map No.	Stream Project Location CE Office Year Completed	Erosion Agent	Protection Method	Present Condition and Remarks
21	Floyd River Sioux City, IA Omaha 1966	Channel realignment would result in a steeper bed gradient and higher flow velocities; grade control was necessary to prevent bed degradation and bank failure	Sheet piling and rock sills (design based on extensive model tests at the University of Iowa by CE personnel)	Very good. Channel relocated
22	West Fork Ditch Onawa, IA Omaha 1972	Channel realignment resulted in a steeper bed slope and higher flow velocities; grade control was necessary to prevent bed degradation and bank failure	Low rock sills in channel bottom; repairs (based on limited model studies at Mead Hydraulic Laboratory) consisted of creating positive sheet-pile crest and short length of rock toe	Good. Extensive erosion during high flows of 1973; no damage thereafter
23	Connecticut River Hanover, NH New England 1962	High stage streamflow through relatively straight reach, water-level fluctuation, freeze-thaw, ice action, and boat wake waves	Stone riprap revetment	Very good. Property is owned by Dartmouth University. Revetment constructed by New England Power Company
24	Connecticut River Thetford, VT New England 1972	High stage streamflow through relatively straight reach, water-level fluctuation, freeze-thaw, ice action, and boat wake waves	Used-tire bulkhead	Very good. Constructed by local resident
25	Connecticut River Turners Falls Pool, MA New England 1977	Water-level fluctuation, freeze-thaw, ice action, high stage flow, and boat wake waves	Tree removal, hydroseeding with and without riprap toe protection	Very good with toe protection, poor without. Nine miles of river bank protected by Northeast Utilities; project has not been tested by high flow
26	Hayward Creek Quincy, MA New England 1977	High stage streamflow through relatively straight reach and overbank flow	Paving block (Monoslab)	Very good. Some minor settling from overbank flow in 1978
27	Winooski River North Williston, VT New York Late 1930's	High stage streamflow against concave bank of bendway, ice action, debris	Stone riprap revetment and rock-filled log cribbing	Good. Poplar log cribs rotted in 4 years; stone and vegetation providing good protection
28	St. Marys River Mission Point, MI Detroit 1974	Wave action	Stone riprap revetment	Excellent. Protects bank of recreational island
29	Illinois Waterway Banner Levee, IL Rock Island 1976	Wave action	Stone riprap revetment	Excellent. Protects farmland behind levee
30	Bureau Creek Bureau County, IL Rock Island 1974	High stage streamflow against concave bank of bendway	Kellner jacks	Fair. Protects levee of I&M Canal; jacks failing

(Continued)

Map No.	Stream Project Location CE Office Year Completed	Erosion Agent	Protection Method	Present Condition and Remarks
31	Iowa River Louisa County, IA Rock Island 1976	High stage streamflow through relatively straight reach	Timber spur jetties	Failed. Protected pipeline; failed due to flanking
32	Minnesota River Savage, MN St. Paul 1966	High stage streamflow against concave bank of bendway and water-level fluctuations caused by passing commercial vessels	Quarry-run stone	Very good. Minor erosion also due to seepage and frost action
33	Minnesota River Mankato, MN St. Paul 1971/79	High stage flow through relative straight reach	Stone riprap revetment of two gradations	Very good. Comparison of quarry-run with well-graded stone
34	Tanana River Fairbanks, AK Alaska 1977/78	Ice action and high stage streamflow through relatively straight reach	Tree revetment, timber mattress, ice revetment	Failed. Failure has occurred on sections of all three methods
35	Fisher River Libby, MT Seattle 1967	Channel realignment resulted in a steeper bed slope and higher flow velocities; grade control was necessary to prevent bed degradation and bank failure	Grade-control structures with stone riprap revetment on side slopes	Good. Channel realignment was necessary to accommodate relocated railroad main line
36	Hocking River Athens, OH Huntington 1971	Channel realignment required side-slope protection, and overbank drainage control	Gravel blanket, stone riprap revetment, crown vetch, drainage interceptor system	Very good. To stabilize channel relocation project in the Hocking River floodplain
37	Ohio River Cloverport, KY Louisville 1973	Seepage, water-level fluctuations, wave action	Stone riprap revetment	Very good. Protects highway
38	Ohio River Newburgh, IN Louisville 1976	High stage streamflow against concave bank of bendway, wave action, seepage	Stone riprap revetment	Very good. 6200 ft of bank protection
39	White River Levee Unit 8, Edwardsport, IN Louisville 1975	High stage flow against concave bank of bendway	Channel cutoff	Good. To protect agricultural levees constructed in 1940
40	Monongahela River California, PA Pittsburgh 1977	High stage streamflow through relatively straight reach; drawdown effect from high water	Coarse-rock-filled used-tire bulkhead	Good. 90 ft of bank protection by local resident
41	Ohio River Wheeling, WV Pittsburgh 1971	High stage streamflow through relatively straight reach; drawdown effects, overbank drainage	Stone riprap revetment on filter fabric	Good. Bank protection at municipal parking garage. Some repair required
42	Ohio River Tiltonsville, OH Pittsburgh 1968	High stage streamflow through relatively straight reach; overbank drainage	Gravel blanket (3/8- to 4-1/2-in. aggregate, no bedding)	Very good. 2600 ft of bank protection

(Continued)

Exhibit 5 (Concluded)

Map No.	Stream Project Location CE Office Year Completed	Erosion Agent	Protection Method	Present Condition and Remarks
43	Woodcock Creek Saegertown, PA Pittsburgh 1973	High stage streamflow against concave bank of bendway	Gabion spurs	Good. Experienced some damage
44	Little Rockfish Creek Hope Mills, NC South Atlantic 1976	High stage flow against concave bank of bendway; seepage	Gabions and vegetation	Good. 20 lin ft of gabions slipped 6-8 ft vertically due to groundwater seepage; repaired with crushed stone and timber toe
45	Mill Creek Mill Creek Levee, CA Los Angeles 1970	High stage streamflow against concave bank of floodway	Gabion midfloodway barrier	Very good. Extremely high velocity and heavy debris
46	Rio Grande River Espanola, NM Albuquerque 1951	High stage streamflow through a relatively straight reach	Kellner jacks, trees	Very good. Minor repairs; protects irrigation canal
47	Cuchillo Negro Creek Truth or Consequences, NM Albuquerque 1977	Streamflow over highly erodible bank soil	Gabion spur dikes and revetment	Very good. Levee protection
48	Trinity River Moss Hill, TX Galveston 1966	High stage streamflow against concave bank of bendway	Timber fence dikes	Good. Protects bridge abutment
49	Arkansas River Merrisach Lake, AR Little Rock 1972	Wave action	Timber pile wall	Very good. In 1980 about 10 percent of wall required some repair to cap boards
50	Arkansas River Ellinwood, KS Tulsa 1974	High stage streamflow against concave bank of bendway	Kellner Jack Fields at four sites	Excellent. Project exposed to major flood in June 1981

(Sheet 5 of 5)

ST. FRANCIS RIVER
CLARK'S CORNER, ARKANSAS

Streambank Erosion Control Evaluation and Demonstration Act of 1974
Section 32 Program - Work Unit 2

EVALUATION OF EXISTING BANK PROTECTION WORKS

(1) Location

Stream St. Francis River Floodway River Mile 49 Side Right

Local Vicinity Clark's Corner Cutoff Lat N35Deg09 Long W 90Deg 39'

At/Nr City Forrest City County St.Francis State AR Cong Dist 1

CE Office Symbol LMMED-DR Responsible Agency Corps of Engineers

Site Map Sources COE 668 Federal Bldg., Memphis, Tenn. 38103

Land Use Farming

(2) Hydrology at or Near Site

Stage Range 1.2 to 39.0 ft; Period of Record 1935 to 1979.

Discharge Range 300 to 49,400 cfs; Velocity Range 1.0 to 5.0 fps

Sediment Range - to - tpd; Period of Record 19 - to 19 -.

Bank-full Stage 24 ft; Flow 16,500 cfs; Average Recurrence Interval 1 yr

Bank-full Flow Velocity: Average 2.5 fps; Near Bank - fps

Comments Data not available at site. Data above is for gage location (Riverfront) approximately 10 miles upstream. No significant flow enters the channel between Riverfront and Clark's

(3) Geology and Soil Properties

Bank (USCS) Upper 7-9ft. (ML), next 10-12'(CL,CH) Bed (USCS) Not available

Data Sources Soil Borings

Groundwater Bank Seepage Insignificant amount

Overbank Drainage Insignificant amount

Comments

(4) Construction of Protection

Need for Protection Bridge abutment endangered by scour pocket which cut into the roadway on downstream side of bridge.

Erosion Causative Agents Eddies and current action due to flow around bridge piers.

Protection Techniques Stone on lumber mattress underwater and filter cloth in dry.

General Design Slope restored by backfilling with sand. Stone was then placed on lumber matresses with cribs underwater and stone was placed on filter fabric in the dry.

Project Length 140 ft; Construction Cost $ $76,398 Mo/Yr Completed Nov 64

(5) Maintenance

Experienced Flows (Stage, cfs, Date) (38.6, 49,400, 4/28/73)
(35.8, 39,300, 4/11/75)

Repairs and Costs (Item, Cost, Data) None

Comments: Experienced flows at Riverfront (2)

(6) Performance Observations and Summary

Monitoring Program Onsite inspection last 3 years.

Documentation Sources Memphis District, Corps of Engineers

Project Effect on Stream Regime Nothing significant

Project Effect on Environment Nothing adverse

Successful Aspects No scour damage since completing construction.

Unsuccessful Aspects High cost and lumber mattress is labor intensive.

General Evaluation This technique was successfully used here and could be used at other locations where uniform riprap placement is desired underwater and where it is critical that the grade and slope of the original embankment be restored to prevent failure of a major structure.

Recommendations

(7) Additional Information, Comments, and Summary

Map No. 1. No additional monitoring of this site is needed. No change is noted from last year's onsite inspection and the only visible change in the site since construction was completed in 1964 is the amount of vegetation. The site experienced record high stages and discharges in 1973.

Attached Items:
1 - 1 - Unique project features and evaluation
1 - 2 & 3 - Project plan and typical cross sections
1 - 4 - Photograph (1963) before construction
1 - 5 - Photographs (1979) after construction

UNIQUE PROJECT FEATURES

This bridge is located in an isolated, little traveled rural area; however, the erosion project is very typical. The type of construction used at this site is considered unique. In 1964 when this project was constructed, filter fabric was seldom used and little was known about it. Also, lumber mattresses with cribs were little used. The combination of the two for the purpose of protecting the sand fill make this an unusual project.

In repairing the damaged bridge, it was necessary to place revetment around the exposed abutments and to extend the protection downstream to prevent further damage to the roadway (See Plan). In order to provide suitable slopes it was necessary to restore some of the slopes by filling. Below the water surface, this was accomplished either by stone fill or sand fill protected by lumber mattress. Above water, or above an established elevation designated as the construction reference plane (CRP), sand fill was used. All upper bank areas, including the sand fill and graded bank, were then paved with 12-inch-thick riprap paving. Plastic filter fabric was used in lieu of a gravel blanket under the riprap paving over the entire sand fill area and on the graded banks up to an elevation necessary to cover sand strata in the bank. The type of filter fabric used was Poly-Filter X. It was placed in 6- and 12-foot strips parallel to water's edge, overlapped 8 inches and fastened down with securing pins at 3-foot intervals.

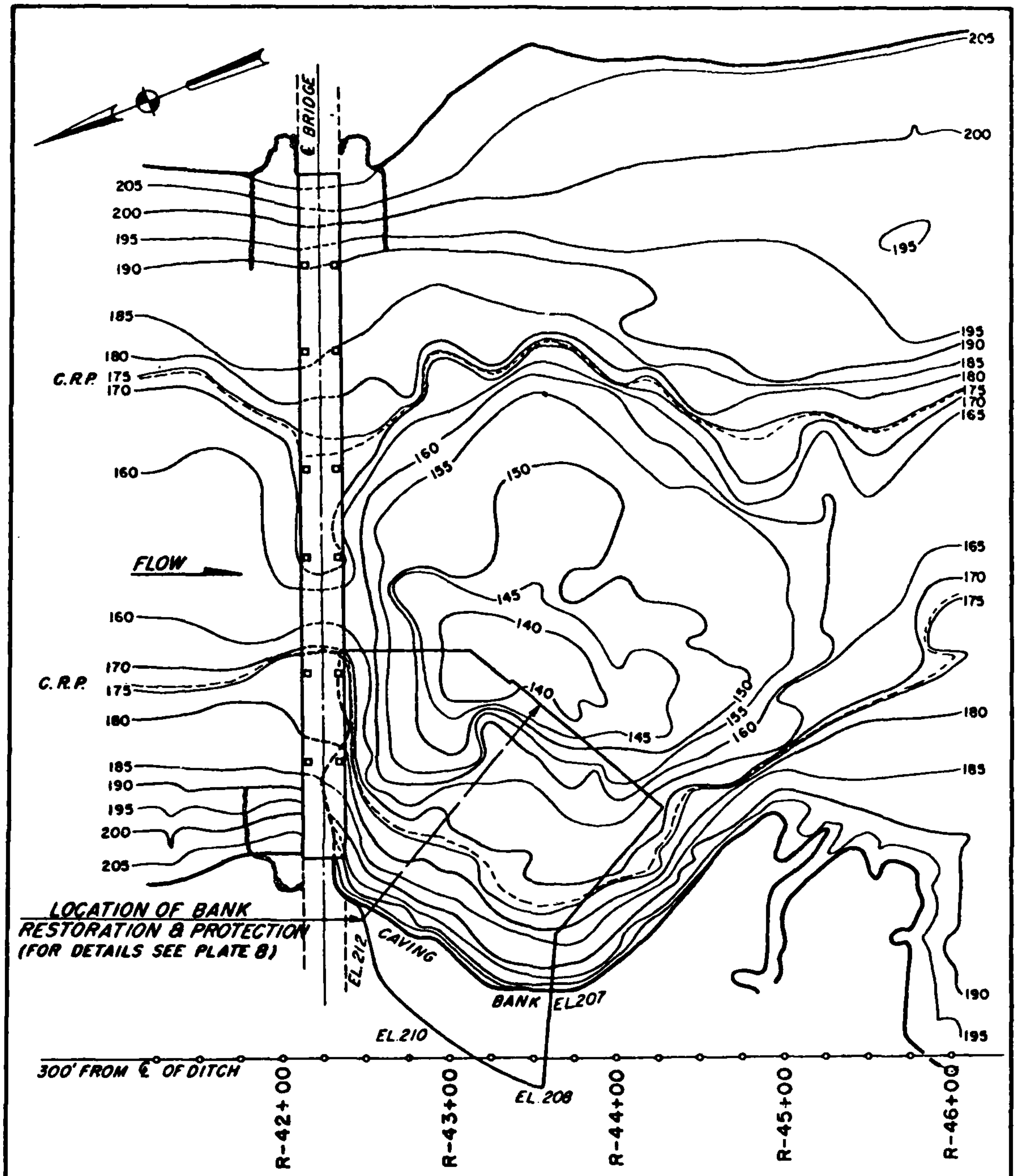
Inspections have been made after high river stages and practically no change has been noted. There has been no indication of loss of material through the paving and no settlement of the paving.

EVALUATION

The streambank protection has been subjected to severe current attack during extremely high flows, and has withstood them without any failure. The initial cost for the project was high, but very little maintenance cost has been required. The most important consideration in repair of a scour hole such as this is that the protection doesn't fail and result in loss of the bridge and possibly loss of lives. This objective has been accomplished at this site and the only additional costs incurred have been for spraying to kill vegetation as required.

No futher monitoring seems necessary at this site. Visually it looks no different from typical stone protection at a bridge site.

This type of construction has been used at other locations in the Memphis District and has been successful.



NOTE: C.R.P. = 174.0 M.S.L.
SCALE 1" = 100'

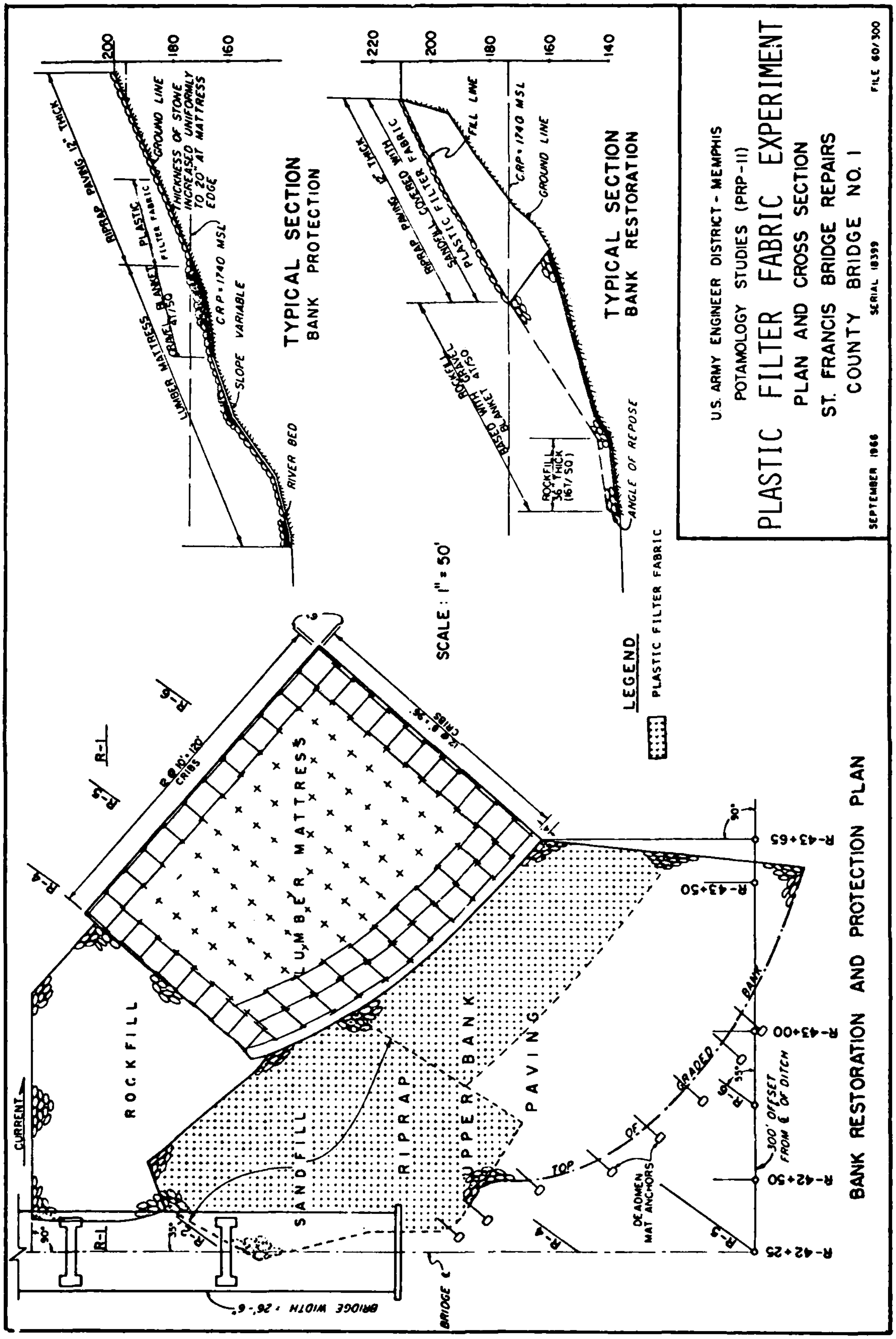
U. S. ARMY ENGINEER DISTRICT - MEMPHIS
POTAMOLOGY STUDIES (PRP - 11)

PLASTIC FILTER FABRIC EXPERIMENT

GENERAL MAP BEFORE REPAIRS
CLARK'S CORNER CUT-OFF, ARK.
COUNTY BRIDGE NO. 1

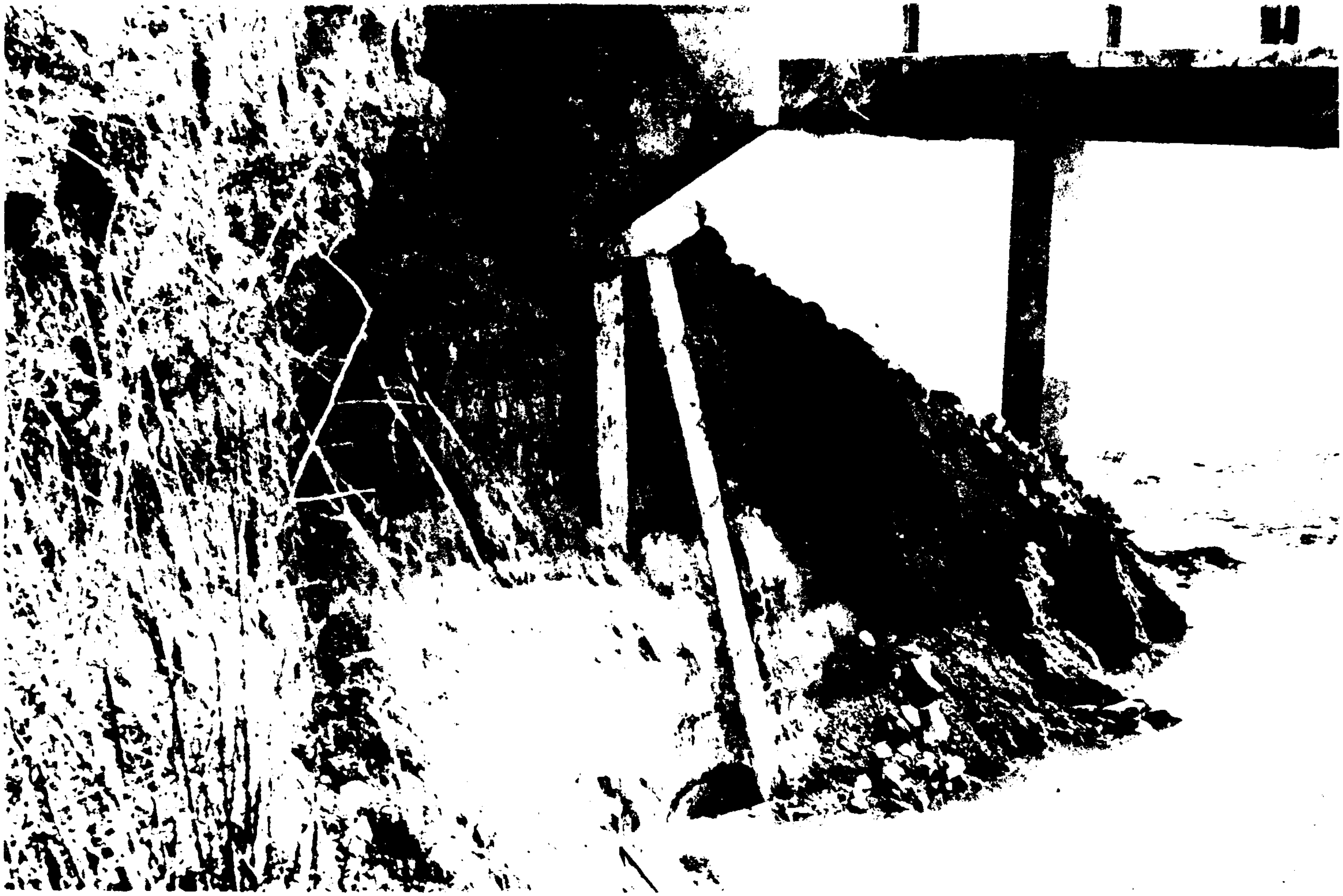
SEPTEMBER 1966 SERIAL 18399 FILE 60/299

ITEM 1-2



U.S. ARMY ENGINEER DISTRICT - MEMPHIS
POTAMOLOGY STUDIES (PRP-II)
PLASTIC FILTER FABRIC EXPERIMENT
PLAN AND CROSS SECTION
ST. FRANCIS BRIDGE REPAIRS
COUNTY BRIDGE NO. 1
SEPTEMBER 1966
SERIAL 18399
FILE 60/300
TYPICAL SECTION
BANK PROTECTION
RIPRAP PAVING 12" THICK
PLASTIC FILTER FABRIC
GROUND LINE
THICKNESS OF STONE INCREASED UNIFORMLY TO 20" AT MATTRESS EDGE
LUMBER MATTRESS
GRAVEL BLANKET
C.R.P. = 174.0 MSL
SLOPE VARIABLE
RIVER BED
TYPICAL SECTION
BANK RESTORATION
FILL LINE
GROUND LINE
ANGLE OF REPOSE
200
180
160
220
140
SCALE: 1" = 50'
LEGEND
PLASTIC FILTER FABRIC
ROCKFILL
SANDFILL
RIPRAP
UPPER BANK
PAVING
LUMBER MATTRESS
CURRENT
DEADMEN MAT ANCHORS
300' OFFSET FROM ℄ OF DITCH
TOP OF GRADED BANK
R-42+25
R-42+50
R-43+00
R-43+50
R-43+65
BRIDGE ℄
BRIDGE WIDTH = 26'-6"
BANK RESTORATION AND PROTECTION PLAN

ITEM 1-3

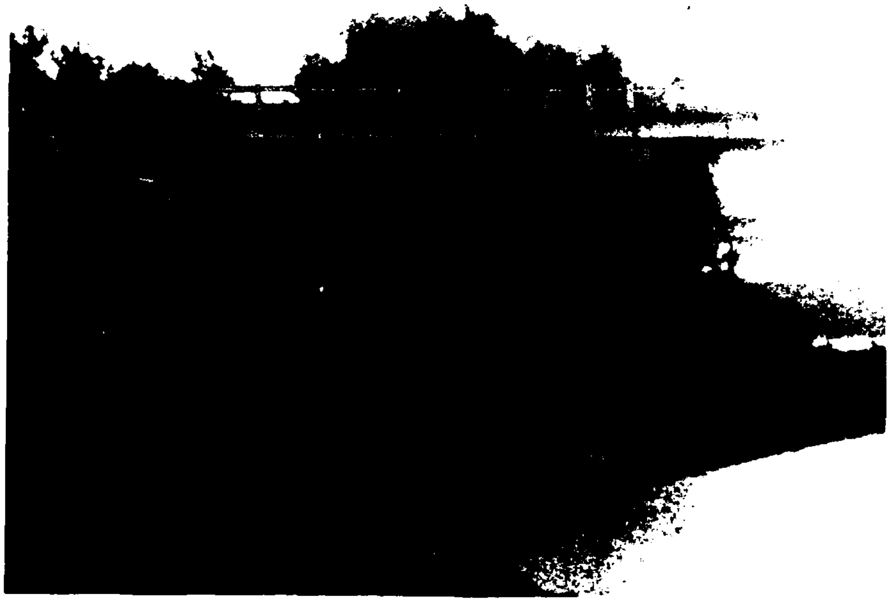


BEFORE CONSTRUCTION (1963)

Scour pocket where active caving was endangering downstream side of west abutment of County Bridge No. 1, Clarks Corner Cutoff, St. Francis Basin Project.

ITEM 1-4

CLARK'S CORNER
1979 PHOTOGRAPHS



LOOKING AT PROTECTION FROM DOWNSTREAM

LOOKING AT PROTECTION FROM BRIDGE

ITEM 1-5

CANEY CREEK
WYNNE, ARKANSAS

Streambank Erosion Control Evaluation and Demonstration Act of 1974
Section 32 Program - Work Unit 2

EVALUATION OF EXISTING BANK PROTECTION WORKS

(1) Location

Stream Caney Creek River Mile (1)* Side L & R

Local Vicinity Watershed Ditch No. 1 Lat N 35° 17' Long W 90Deg 50'

At/Nr City Wynne County Cross State AR Cong Dist 1

CE Office Symbol LMMED-DR Responsible Agency USDA, Soil Conservation Service

Site Map Sources Soil Conservation Service, Wynne, Arkansas

Land Use Farming

(2) Hydrology at or Near Site

Stage Range – to – ft; Period of Record 19– to 19–.

Discharge Range – to – cfs; Velocity Range – to – fps

Sediment Range – to – tpd; Period of Record 19– to 19–.

Bank-full Stage – ft; Flow – cfs; Average Recurrence Interval – yr

Bank-full Flow Velocity: Average – fps; Near Bank – fps

Comments Hydrology Information Unavailable

(3) Geology and Soil Properties

Bank (USCS) Upper 15-17 ft.(ML),18-55ft (ML or CL) Bed (USCS) Not available

Data Sources Soil Borings

Groundwater Bank Seepage Insignificant amount

Overbank Drainage Insignificant amount

Comments

(4) Construction of Protection

Need for Protection Severe eroding banks resulted in loss of valuable farmland and development of inefficient drainage channels.

Erosion Causative Agents Type of soil (dispersive clay) is very susceptible to erosion when in contact with flowing water.

Protection Techniques Lime and gypsum treatment, clay gravel lining and vegetation

General Design Slopes graded to 1V on 3H. Reach divided into plats and different treatments of lime, gypsum, clay gravel and vegetation were applied.

Project Length 1,700 ft; Construction Cost $ $38,800 Mo/Yr Completed Sep 75

*(1) Ditch No. 1, Sta. 494+00 - 511+00

(5) Maintenance

Experienced Flows (Stage. cfs. Date) Bankfull flows occur almost every year during the rainy season.

Repairs and Costs (Item. Cost. Data) None

Comments: Maintenance consists of mowing and removal of small trees and brush.

(6) Performance Observations and Summary

Monitoring Program SCS is monitoring in addition to our on site inspections

Documentation Sources Soil Conservation Service

Project Effect on Stream Regime Bank erosion and caving have been stopped, resulting in a much more efficient channel.

Project Effect on Environment Nothing adverse; reduced erosion; vegetation provides good wildlife habitat.

Successful Aspects Erosion virtually stopped, channel much more efficient and looks much better.

Unsuccessful Aspects High costs

General Evaluation All types of treatment tried in this project could be successfully used in treating dispersive clay.

Recommendations These types of treatment should be considered when designing channels in dispersive clays.

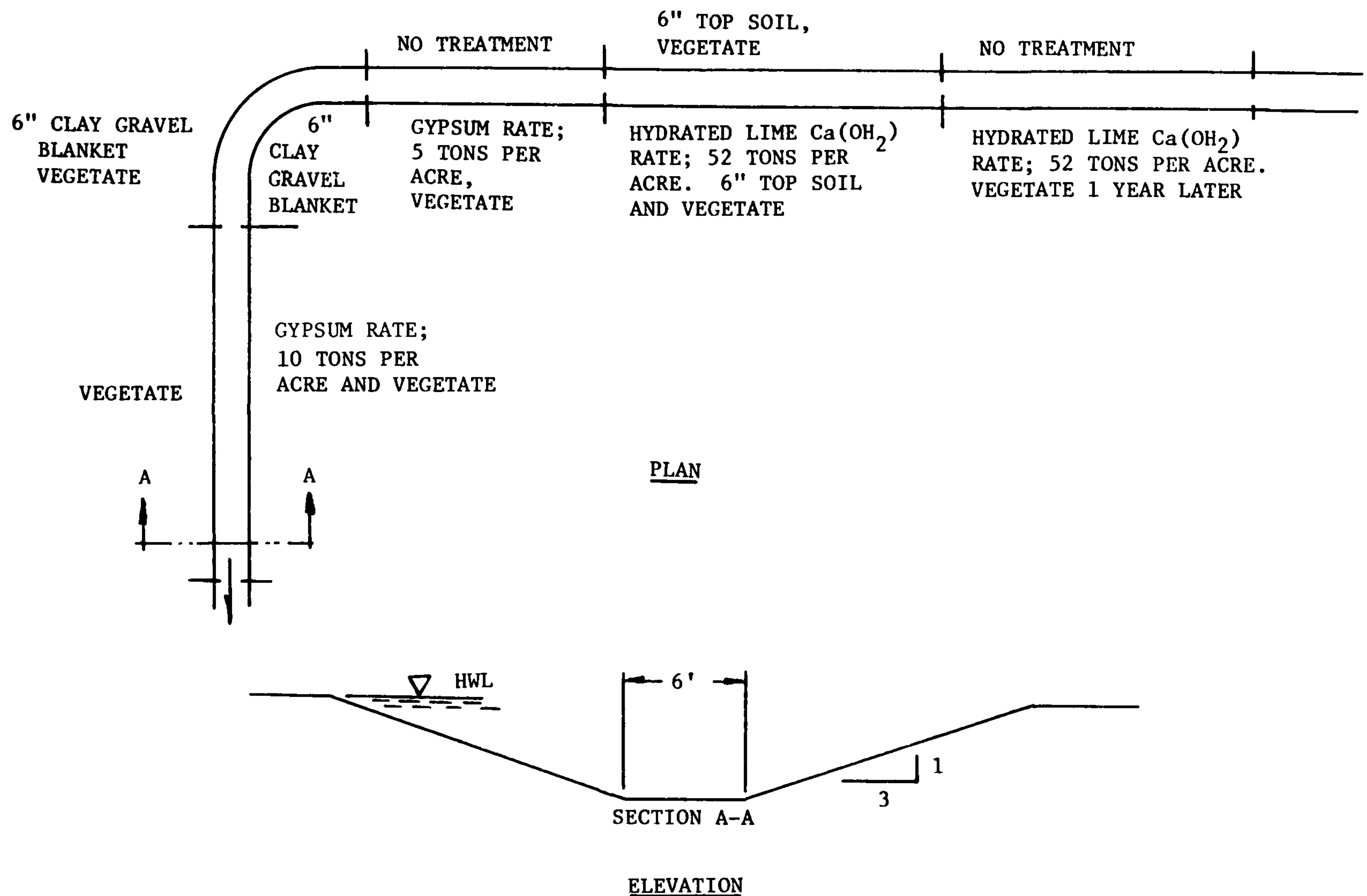
(7) Additional Information, Comments, and Summary

Map No. 2. SCS will monitor this project through 1979.

No additional monitoring is needed at this site. Before, during and after construction photographs of the site were submitted with the 1978 monitoring report.

Attached Items:

2 - 1 - Project Plan & typical cross section
2 - 2 - Unique project features and evaluation
2 - 3 - Photographs before and after construction
2 - 4 - Photographs during and after design flow


NO TREATMENT
6" TOP SOIL, VEGETATE
NO TREATMENT
6" CLAY GRAVEL BLANKET VEGETATE
6" CLAY GRAVEL BLANKET
GYPSUM RATE; 5 TONS PER ACRE, VEGETATE
HYDRATED LIME Ca(OH2) RATE; 52 TONS PER ACRE. 6" TOP SOIL AND VEGETATE
HYDRATED LIME Ca(OH2) RATE; 52 TONS PER ACRE. VEGETATE 1 YEAR LATER
GYPSUM RATE; 10 TONS PER ACRE AND VEGETATE
VEGETATE
A
A
PLAN
HWL
6'
1
3
SECTION A-A
ELEVATION

ITEM 2-1

UNIQUE PROJECT FEATURES

This channel and many others in the surrounding area were designed for drainage of agricultural land. The channel is serving the purpose of the original design but because of dispersive soil the farmland is eroding into the channel and creating a real "eyesore."

The main consideration in the design of this trial treatment project was slope protection. Experience has indicated that 1V-on-3H slopes were probably best suited for this type of soil, but other treatment was likely needed because the soil is highly dispersive due partly to a high sodium content.

A 1,500-foot section of channel was selected and divided into ten 300-foot plots and treated as shown on the attached plan. The hydrated lime and gypsum were incorporated into the soil by disking and where vegetation was called for, half of the plot was sprigged with common bermuda and half with coastal bermuda.

All of the plots have performed well since construction. Some erosion is occurring in the plot which was graded only, but the erosion is not significant at the present time.

The only maintenance performed on the test plots thus far has been mowing after the first year. Since the side slopes are so flat, the channel may be easily maintained by mowing.

EVALUATION

Channels in agricultural land need to be designed for protection against erosion although they may not be seen much by the public. This project demonstrates several types of treatment which may be used to prevent erosion of channels in dispersive soils and thus enhance the surrounding land.

The initial cost of the project was very high ($38,000). The cost of some of the treatments would probably prevent them from being used except maybe in small isolated areas. The maintenance costs have been and should continue to be small since the channel may be mowed.

Since a monitoring program is being conducted by the SCS, there will be no need for additional monitoring.

Any of these types of treatment could be used in the future. It would have been good to have attempted vegetating some of the plots by seeding to see if a good stand of vegetation could be obtained by this method also.

ITEM 2-2

BEFORE CONSTRUCTION LOOKING DOWNSTREAM FROM UPPER END

AFTER CONSTRUCTION LOOKING DOWNSTREAM FROM BEND

PHOTOGRAPHS BEFORE AND AFTER CONSTRUCTION

ITEM 2-3

LOOKING DOWNSTREAM AT LOWER END DURING HIGH FLOW

LOOKING DOWNSTREAM FROM UPPER END AFTER DESIGN FLOW

ITEM 2-4

RED RIVER
MORAMEAL, LOUISIANA

Streambank Erosion Control Evaluation and Demonstration Act of 1974
Section 32 Program - Work Unit 2

EVALUATION OF EXISTING BANK PROTECTION WORKS

(1) Location

Stream Red River, Louisiana River Mile 257 Side Left

Local Vicinity Morameal Lat $32^{o}21'$ Long 93^{o} 35'

At/Nr City Shreveport, LA County Bossier State LA Cong Dist 4

CE Office Symbol LMNED-DR Responsible Agency Corps of Engineers

Site Map Sources Corps of Engineers, P.O. Box 60267, New Orleans, LA 70160

Land Use Farming, oil production

(2) Hydrology at or Near Site

Stage Range 1 to 30 ft; Period of Record 1963 to 1980.

Discharge Range 690 to 303,000 cfs; Velocity Range .5 to 7 fps

Sediment Range 301 to 1,600,639 pd; Period of Record 1963 to 1979.

Bank-full Stage 140 ft; Flow 70,000 cfs; Average Recurrence Interval 1 yr

Bank-full Flow Velocity: Average 4.57 fps; Near Bank .5 fps

Comments Stages ref. Shreveport gage, zero = 131.5 NGVD

(3) Geology and Soil Properties

Bank (USCS) Silt, clay, primarily high erodible sand Bed (USCS) Sand, silt, clay

Data Sources Corps of Engineers, F&M Branch

Groundwater Bank Seepage None observed

Overbank Drainage From rainfall runoff

Comments Proper ditching along upper bank would have eliminated overbank drainage and resulting problems.

(4) Construction of Protection

Need for Protection Arrest riverbank erosion, maintain integrity of levee system

Erosion Causative Agents High-velocity attack in concave bend during high river stages

Protection Techniques See Attachment 1, page 1

General Design Standard trenchfill revetment section, substituting various materials in lieu of standard Type C Stone

Project Length 7,100 ft; Construction Cost $1,901,544 Mo/Yr Completed Nov 75

(5) Maintenance

Experienced Flows (Stage, cfs, Date) High stage of 22.5 feet in April 1979

Repairs and Costs (Item, Cost, Data) See Attachment 1, page 2

Comments: No major maintenance to date.

(6) Performance Observations and Summary

Monitoring Program Periodic on-site inspection

Documentation Sources Corps of Engineers

Project Effect on Stream Regime Erosion stopped

Project Effect on Environment Reduced bank erosion and sediment input

Successful Aspects Stabilization affected, alignment preserved, levee system protected.

Unsuccessful Aspects See Attachment 1, page 2

General Evaluation Overall cost competitive, methods provide fuel and transportation savings.

Recommendations The first high water experienced at this location was in April 79. Several more high-water seasons will be necessary for proper evaluation all experimental sections.

(7) Additional Information, Comments, and Summary

Map No.3. Gobi mat provided the most attractive aesthetic results, with gabions second.

Attached Items:

3 - 1 - Protection Techniques

3 - 2 - Project plan and typical section

3 - 3 - Photographs before, during and after construction

3 - 4 - Photographs 2½ years after construction

Protection Techniques. This revetment was constructed using standard revetment design, divided into seven sections using various construction materials, in upstream order as follows:

1. Standard Trenchfill - 740-foot control sections at downstream end of revetment. This design and standard stone are proven to be effective in erosion control, and was used at the D/S end to insure the revetment would not be flanked.

2. Soil Cement - 1,000-foot section of soil cement blocks in lieu of stone. Layers of soil cement made with batture sand were scored to yield a gradation of sizes similar to standard stone gradation. The blocks were then placed in the revetment structure like standard stone.

3. Sand-filled Bags - 361-foot section of sandbags made of special weave acrylic material designed by Monsanto Textiles, Co. The material was designed to be economical, strong, durable, insensitive to sunlight and tight enough weave to retain the fine batture material used to fill the bags. The sandbags were placed in the standard trenchfill revetment configuration.

4. Stone-filled Gabions - 1,000-foot section of standard trenchfill design, using 6-inch thick gabions for upper bank protection instead of the standard 12- to 21-inch layer of riprap, but retaining standard stone and design for the trench portion. Gabions are rectangular baskets made of galvanized wire, the ones used in this contract having dimensions of approximately 6.5 feet wide by 12 feet long, 6" thick and a mesh size of less than 4". This permits the baskets to be filled with a smaller gradation stone and savings in quantity of upper bank paving stone used.

5. Cellular Concrete Blocks (Gobi Mat) With Underlying Filter Fabric - 1,500-foot section as in section 4, but using gobi blocks for the upper bank paving. The 8-inch by 8-inch by 4-inch thick concrete blocks were manufactured in a plant not onsite, using aggregates available in Louisiana. Plastic filter fabric was placed under the blocks to prevent leaching of bank material from between the block spaces.

6. Cellular Concrete Blocks (Gobi Mat) - 1,500-foot section as in section 5, but without underlying filter fabric. This section is a control to evaluate the effectiveness of filter fabric.

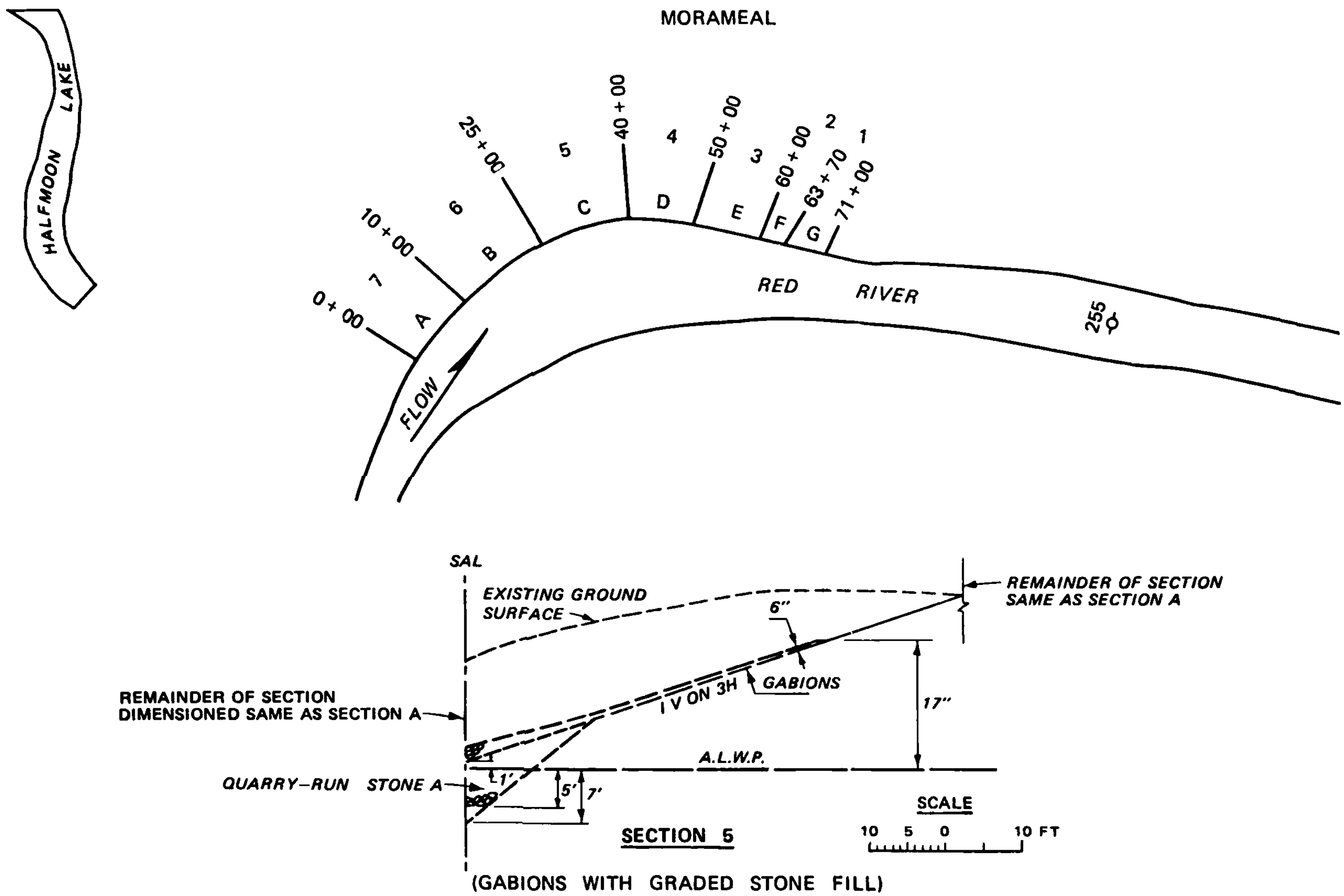
7. Louisiana Stone - 1,000-foot section of standard trenchfill design, using stone available from Louisiana quarry, to test the durability and effectiveness of locally obtainable stone.

ITEM 3-1
(Sheet 1 of 2)

Repairs and Costs (Item, Cost, Date) - In 1976, section 4 was repaired. The contractor assembled the gabions, but used nongalvanized hog rings in an attempt to speed up production, instead of the galvanized tie wire provided and specified in assembly instructions. The hog rings rapidly showed signs of rust and deterioration, and the contractor retied gabions according to design. In 1977 minor filling was done in section 4, because overbank drainage was leaching material from beneath the gabions. No lateral drainage had been provided in this contract. Some sandbags were replaced, damaged presumably by cattle hooves.

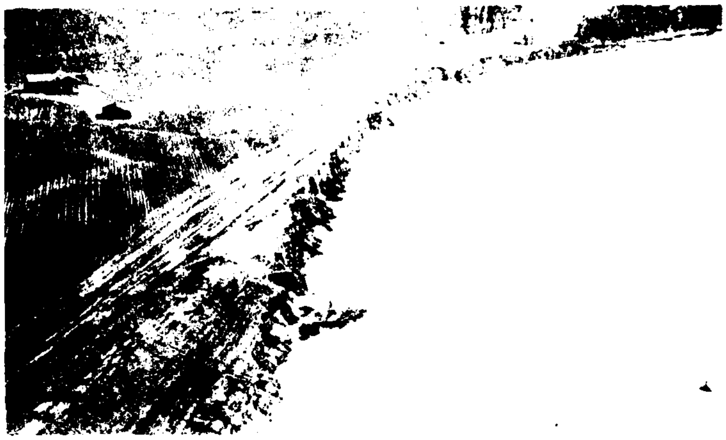
Unsuccessful Aspects Material cost high, compared with riprap. In particular, section 2 shows signs of serious structural problems. The revetment slope and trench shows undesirable bonding between blocks of soil cement, causing the trench section to be unable to conform easily to small areas of erosion and instead it has cracked off in large chuncks, falling riverward and leaving the bank unprotected. The suspected cause is improper mixing, cement-soil content, and/or insufficient curing time before cutting and installation into the revetment section. Section 3 shows signs of reduction of strength of the acrylic bag material, but does not seem attributable to sunlight. The strength loss occurred also in that portion of material against the bank and shielded from light.

ITEM 3-1
(Sheet 2 of 2)



RIVER AT MORAMEAL, LA. PROJECT PLAN AND TYPICAL SECTION

ITEM 3-2



BEFORE CONSTRUCTION

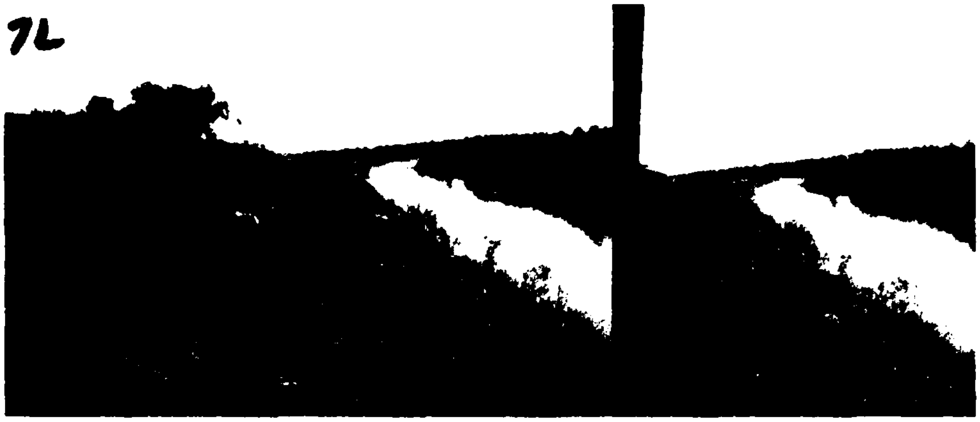
DURING CONSTRUCTION

AFTER CONSTRUCTION

RED RIVER AT MORAMEAL, LA.

ITEM 3-3

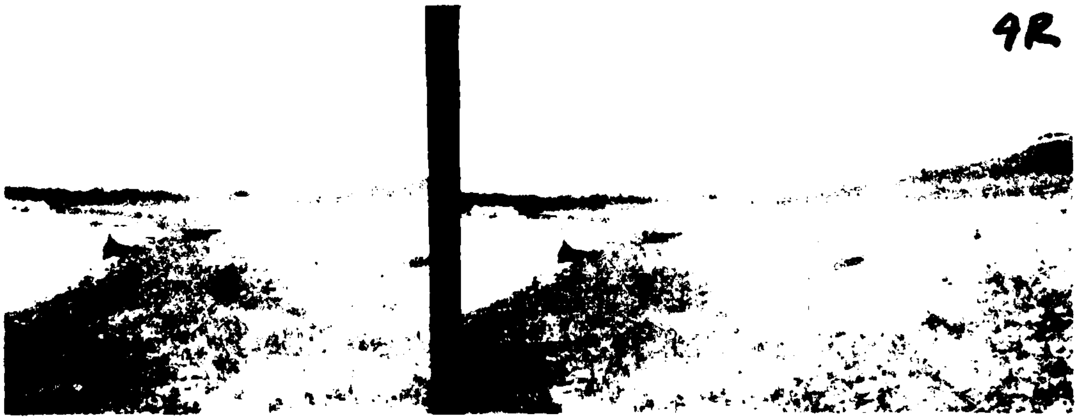
7L



Section 1. Standard Trenchfill Riprap - Looking Downstream

5L

Section 3. Sand-filled Acrylic Bags - Looking Downstream



Section 5. Gabion Mattress with Filter Fabric - Looking Upstream

RED RIVER AT MORAMEAL, LA.
PHOTOGRAPHS 2-1/2 YEARS AFTER CONSTRUCTION

ITEM 3-4

RED RIVER
FAUSSE, LOUISIANA

Streambank Erosion Control Evaluation and Demonstration Act of 1974
Section 32 Program - Work Unit 2

EVALUATION OF EXISTING BANK PROTECTION WORKS

(1) Location

Stream Red River River Mile 178.1 Side Left

Local Vicinity Fausse Lat 31° 47' Long 93° 01'

At/Nr City Natchitoches, LA County Nat. State LA Cong Dist 5

CE Office Symbol LMNED-DR Responsible Agency Corps of Engineers

Site Map Sources Corps of Engineers, P.O. Box 60267, New Orleans, LA 70160

Land Use Farming

(2) Hydrology at or Near Site

Stage Range 5 to 37 ft; Period of Record 1963 to 1979.

Discharge Range 880 to 275,000 cfs; Velocity Range .5 to 8 fps

Sediment Range 301 to 1,600,639 tpd; Period of Record 1963 to 1979.

Bank-full Stage 95 ft; Flow 50,000 cfs; Average Recurrence Interval 1 yr

Bank-full Flow Velocity: Average 4.36 fps; Near Bank .5 fps

Comments Stages ref. Grand Ecore gage, zero = 75.09 NGVD

(3) Geology and Soil Properties

Bank (USCS) Sand, silt, clay Bed (USCS) Sand, silt, clay

Data Sources Corps of Engineers, F&M Branch

Groundwater Bank Seepage N/A

Overbank Drainage By rainfall

Comments No lateral drainage provided in this contract

(4) Construction of Protection

Need for Protection Arrest riverbank erosion

Erosion Causative Agents High stage river currents

Protection Techniques Trenchfill revetment, pile trail dike with stonefill

General Design Trenchfill revetment, transitioning to pile dike to preserve alinement

Project Length 2,200 ft; Construction Cost $ 787,280 Mo/Yr Completed Apr 75

(5) Maintenance

Experienced Flows (Stage, cfs, Date) High stage of 29.96 feet in April 1979

Repairs and Costs (Item, Cost, Data) None

Comments:

(6) Performance Observations and Summary

Monitoring Program Periodic on-site inspection

Documentation Sources Corps of Engineers

Project Effect on Stream Regime Erosion stopped

Project Effect on Environment Reduced bank erosion and sediment input

Successful Aspects Alinement preserved, bank erosion stopped

Unsuccessful Aspects Contractor had some difficulty maintaining correct alinement of pile dike

General Evaluation System performed as intended

Recommendations

(7) Additional Information, Comments, and Summary

Map No. 4. Piling was driven at El. +5' above specification limits, as permitted by contracting officer because of prevailing high river stages

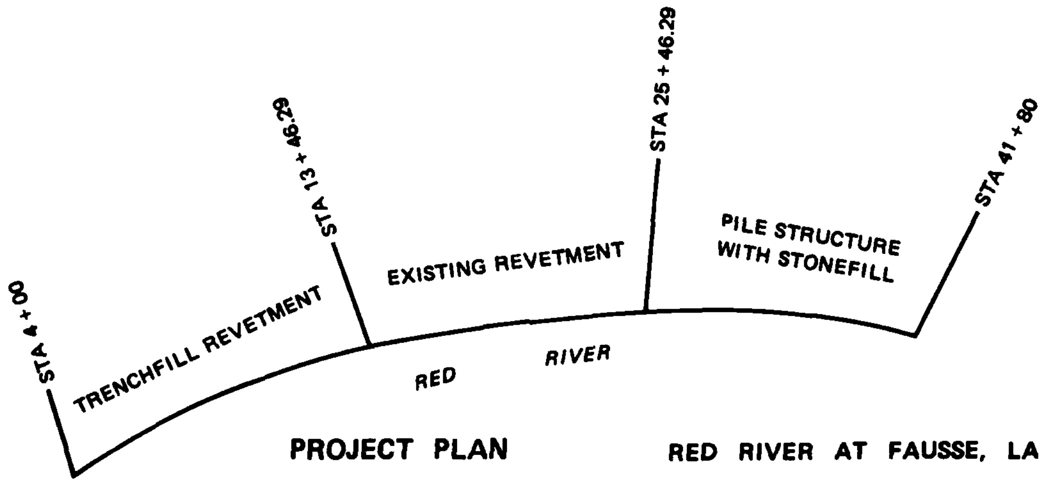
Attached Items:

4 - 1 - Project Summary and Project Plan
4 - 2 & 3 - Protection techniques detail
4 - 4 - Photographs before and after construction

PROJECT SUMMARY

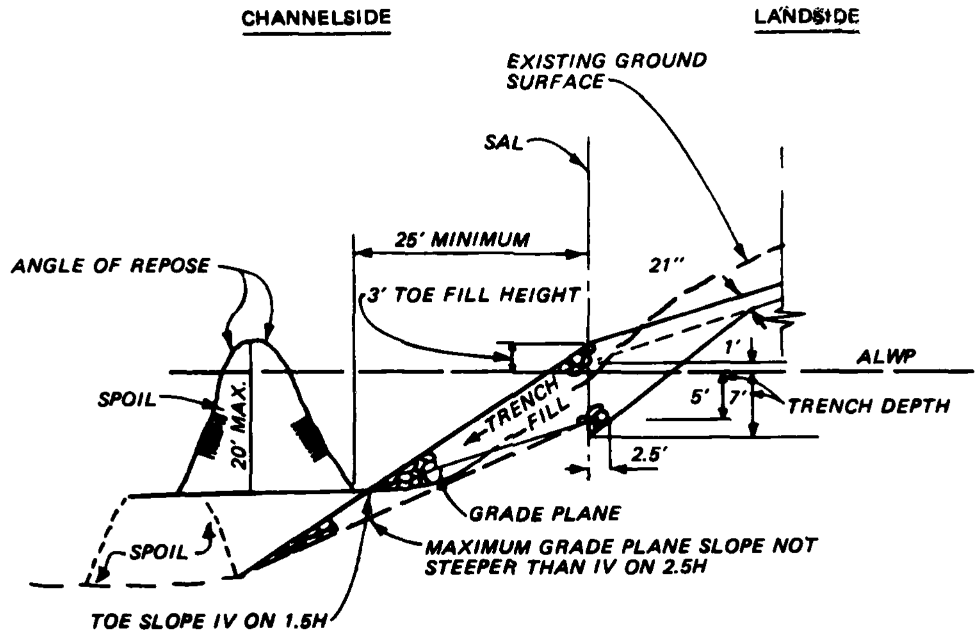
1. EMERGENCY BANK PROTECTION (FAUSSE REVETMENT) WAS PLACED ON THE RED RIVER TO PREVENT THE NECESSITY OF A LEVEE SETBACK AND TO HOLD CHANNEL ALIGNMENT FOR NAVIGATION IN A HIGHLY DEVELOPED AGRICULTURAL AREA. THE REVETMENT CONSISTS OF A 1200-FT LONG MODIFIED TRENCHFILL SECTION (BOTTOM OF TRENCH CONSTRUCTED TO AN ELEVATION OF 14 FT ABOVE INSTEAD OF 5 FT BELOW ALW) CONSTRUCTED IN JULY 1973; IN 1974 A 3100-FT UPSTREAM EXTENSION (PILE REVETMENT) WAS PLACED.

EXTENSION (STANDARD TRENCHFILL) AND A 1600-FT DOWNSTREAM EXTENSION (PILE REVETMENT WITH STONEFILL AND PILE DIKES WITH STONEFILL) WERE CONSTRUCTED. THE 1974 CONTRACT WAS MODIFIED TO INCLUDE REGRADING AND DRESSING OF THE ORIGINAL SECTION WITH THE ADDITION OF STONE TO REPLACE STONE THAT HAD BEEN LAUNCHED AFTER CONSTRUCTION. AFTER CONSTRUCTION OF THE ORIGINAL SECTION BUT PRIOR TO CONSTRUCTION OF THE DOWNSTREAM EXTENSION, CONSIDERABLE DOWNSTREAM FLANKING HAD OCCURRED. THE UPSTREAM EXTENSION PROBABLY PREVENTED ANY UPSTREAM FLANKING THAT WOULD HAVE OCCURRED. THIS EMERGENCY PROTECTION HAS BEEN SUCCESSFUL AND HAS PROVED TO BE AN EXPEDIENT MEANS OF PROTECTING VALUABLE FARMLAND FROM ENCROACHMENT BY THE RED RIVER.

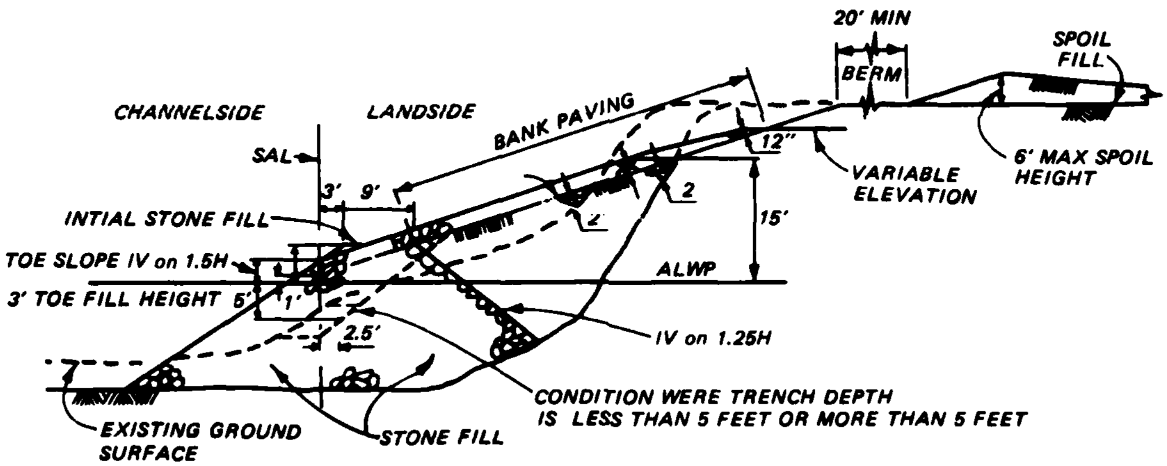


PROJECT PLAN   RED RIVER AT FAUSSE, LA

ITEM 4-1



TYPE A, B, OR D



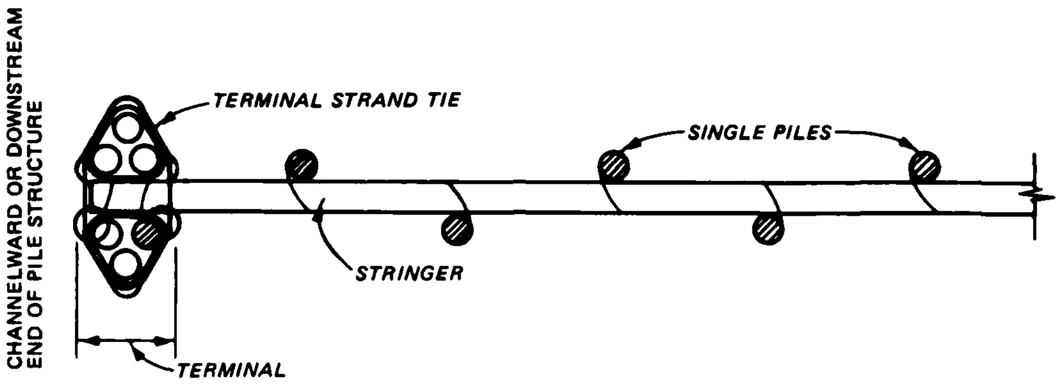
TYPE C

RED RIVER AT FAUSSE, LA

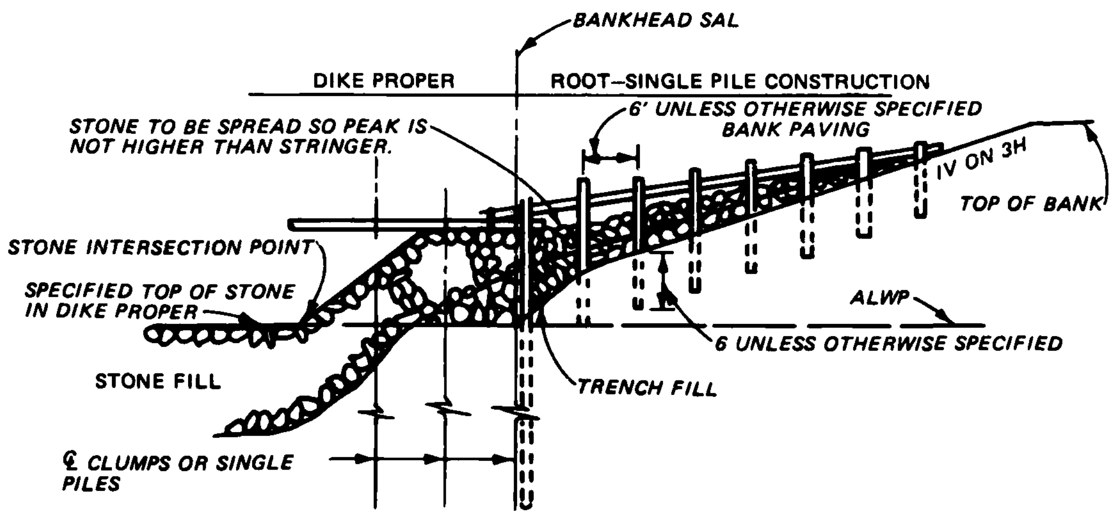
TRENCH FILL REVETMENT

TYPICAL SECTIONS

ITEM 4-2



SINGLE-PILE 2-CLUMP TERMINAL



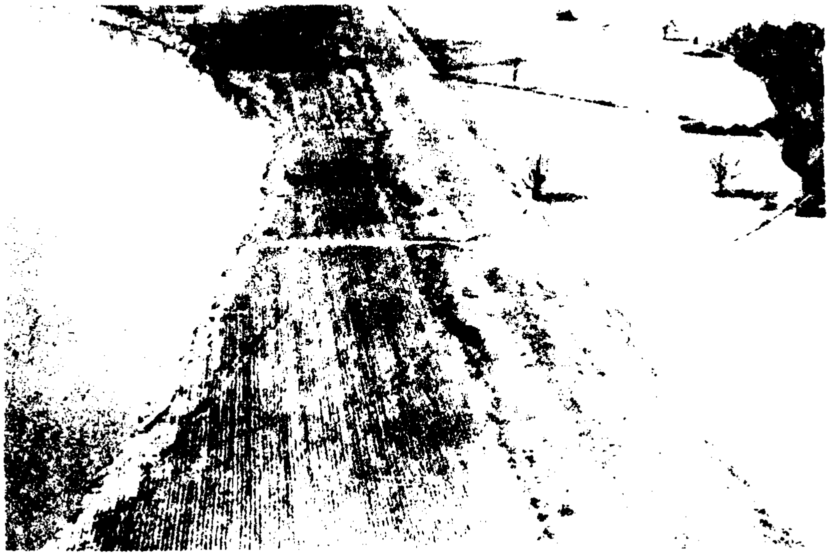
TYPICAL PROFILE PILE ROOT DETAIL

RED RIVER AT FAUSSE, LA

PILE STRUCTURE

TERMINAL, ROOT AND STONE-FILL DETAILS

ITEM 4-3



Aerial view of site before serious bank erosion
occurred which threatened the levee

Aerial view of completed revetment

RED RIVER AT FAUSSE, LA.

ITEM 4-4

RED RIVER
PEROT, LOUISIANA

Streambank Erosion Control Evaluation and Demonstration Act of 1974
Section 32 Program - Work Unit 2

EVALUATION OF EXISTING BANK PROTECTION WORKS

(1) Location

Stream Red River. Louisiana River Mile 187.5 Side Left

Local Vicinity Perot Lat 31° 51' Long 93° 06'

At/Nr City Natchitoches County Nat. State LA Cong Dist 5

CE Office Symbol LMNED-DR Responsible Agency Hold That River, Inc.

Site Map Sources Hold That River, Inc. 1233 W. Loops, Houston, TX (713) 850-3400

Land Use Farming

(2) Hydrology at or Near Site

Stage Range 5 to 37 ft; Period of Record 1963 to 1979.

Discharge Range 880 to 275,000 cfs; Velocity Range .5 to 5.4 fps

Sediment Range 301 to 1,600,639 tpd; Period of Record 1963 to 1979.

Bank-full Stage 115 ft; Flow 161,000 cfs; Average Recurrence Interval 30 yr

Bank-full Flow Velocity: Average 4.8 fps; Near Bank .5 fps

Comments Stages ref. Grand Ecore gage, zero = 75.09 NGVD

(3) Geology and Soil Properties

Bank (USCS) Silt, clay, primarily sand Bed (USCS) Sand, silt, clay

Data Sources Corps of Engineers, F&M Branch

Groundwater Bank Seepage None observed

Overbank Drainage From rainfall runoff

Comments No lateral drainage provided

(4) Construction of Protection

Need for Protection Protection of Black Lake Pipeline, Co., 8" pipeline crossing, prevent bank erosion & restore eroded section of bank.

Erosion Causative Agents High-velocity attack in concave bend during high river stages.

Protection Techniques Fence-like panel jetty dikes

General Design 35' long flexible permeable panel of treated timber, bolted loosely to two 35' pipe rails, driven in bank, axis of structure placed normal to stream flowline.

Project Length 4,000 ft; Construction Cost $ N/A Mo/Yr Completed Apr 70

(5) Maintenance

Experienced Flows (Stage, cfs, Date) High stage of 34.87 feet in May 1973

Repairs and Costs (Item, Cost, Data) Cost not available, major maintenance and U/S extension of protection works in 1978. Minor annual maintenance not performed because of restricted access.

Comments: Fence structures are damaged rather easily by large drift.

(6) Performance Observations and Summary

Monitoring Program Periodic on-site inspection.

Documentation Sources Black Lake Pipeline,Co., P.O. Box 308 Independence, AR 67301

Project Effect on Stream Regime Bank recession halted, lower-bank building

Project Effect on Environment Reduced bank erosion and sediment input.

Successful Aspects Stabliization affected, alignment preserved, pipeline protection accomplished.

Unsuccessful Aspects Expensive to maintain for long-term protection-- 80 percent lost of fence due to flanking.

General Evaluation System performed as intended. Current cost of similar system between $380,000 - $800,000. Anticipated minor maintenance costs, $5,000 per year for first five years, negligible thereafter.

Recommendations Suitable protection for only short term.

(7) Additional Information, Comments, and Summary

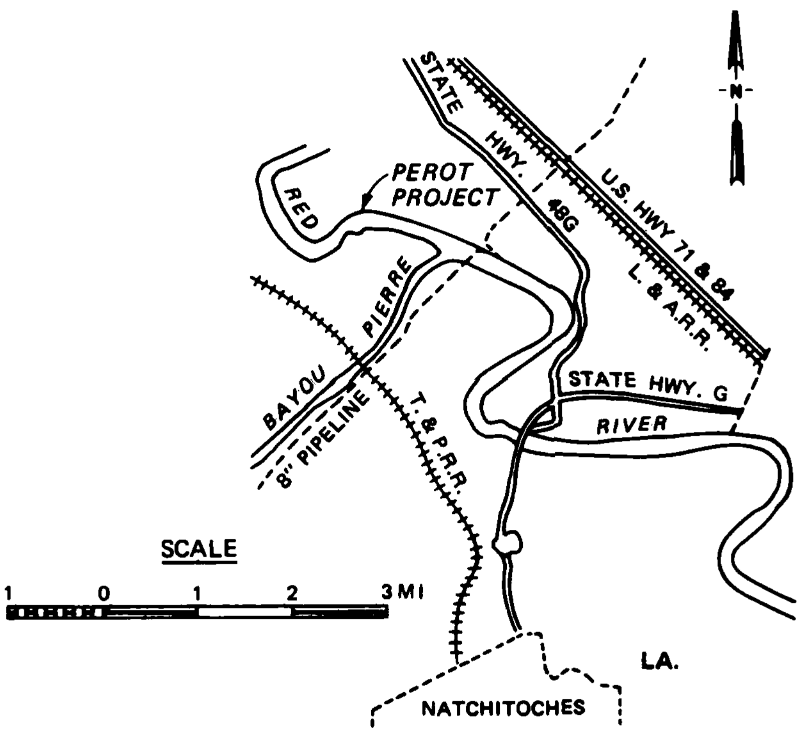
Map No. 5. System is not very attractive in appearance, but controls erosion successfully in specific applications for short term.

Attached Items:

5 - 1 - Project Summary and Location Map
5 - 2 - Project layout and typical section
5 - 3 - Photos before and during and after construction

PROJECT SUMMARY

BETWEEN FEBRUARY AND AUGUST 1970, HOLD-THAT-RIVER ENGINEERING CO., INC., INSTALLED 75 30-FT-LONG PERMEABLE SPUR JETTIES TO PROTECT THE LEFT BANK ALONG A 4000-FT REACH OF THE RED RIVER. THESE JETTY PANELS WERE DESIGNED TO PROVIDE 5 YEARS OF STABILIZATION FOR AN 8-IN. PIPELINE **CROSSING** BELONGING TO BLACK LAKE PIPELINE CO. LOCATED NEAR THE DOWNSTREAM END OF PEROT BEND. APPROXIMATELY 80 PERCENT OF THE JETTY SYSTEM HAS BEEN LOST AS A RESULT OF FLANKING ACTION UPSTREAM FROM THE PROJECT. THE DOWNSTREAM END OF THE JETTY SYSTEM IS STILL FUNCTIONING AND PROVIDING PROTECTION FOR THE PIPELINE CROSSING. NO APPRECIABLE EFFECT HAS BEEN NOTED IN THE CHANNEL UPSTREAM FROM THE JETTY; HOWEVER, THE CHANNEL DOWNSTREAM FROM THE JETTY SYSTEM DID REMAIN STABLE AS LONG AS THE ENTIRE SYSTEM WAS INTACT. SUCH A SYSTEM OF PROTECTION IS SUITABLE WHERE ONLY SHORT-TERM (i.e., LESS THAN 5 YEARS) STABILIZATION IS NEEDED. FOR LONG-TERM PROTECTION, MAINTENANCE COST WOULD BE EXCESSIVE.

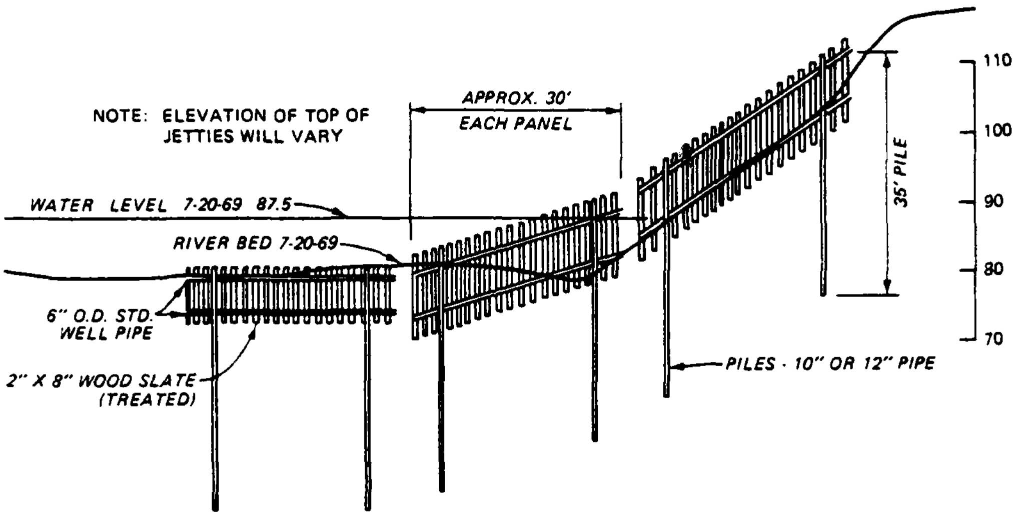


LOCATION MAP

RED RIVER AT PEROT, LA

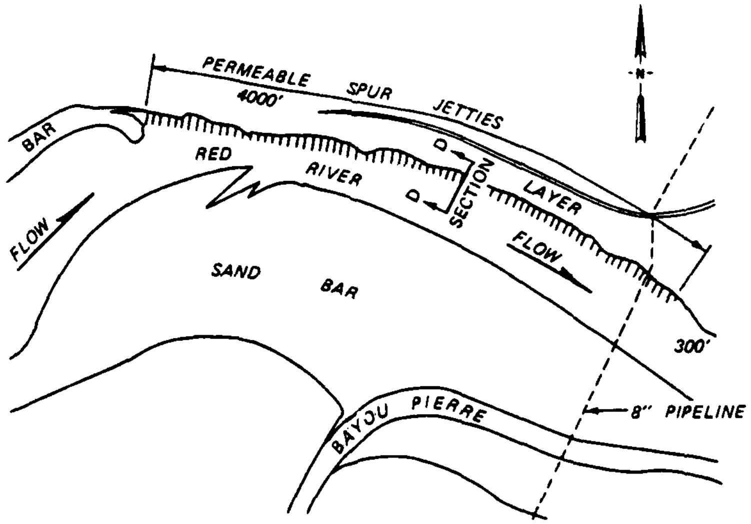
PROJECT SUMMARY AND LOCATION MAP

ITEM 5-1



TYPICAL SECTION

SCALE
HORIZONTAL AND VERTICAL
10 0 10 20 FT



PROJECT LAYOUT

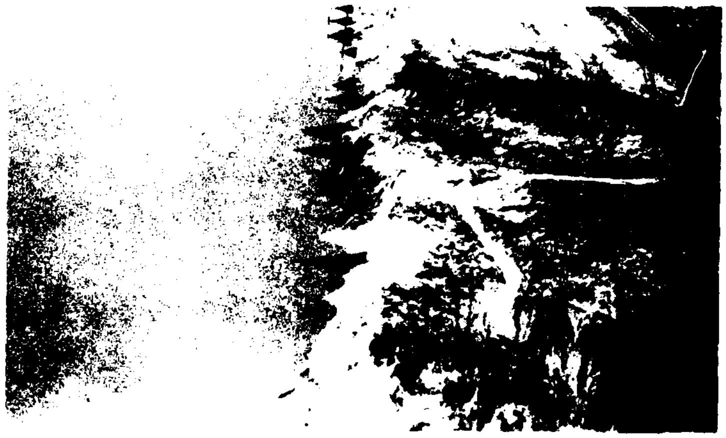
SCALE
1000 0 1000 2000 FT

RED RIVER AT PEROT, LA

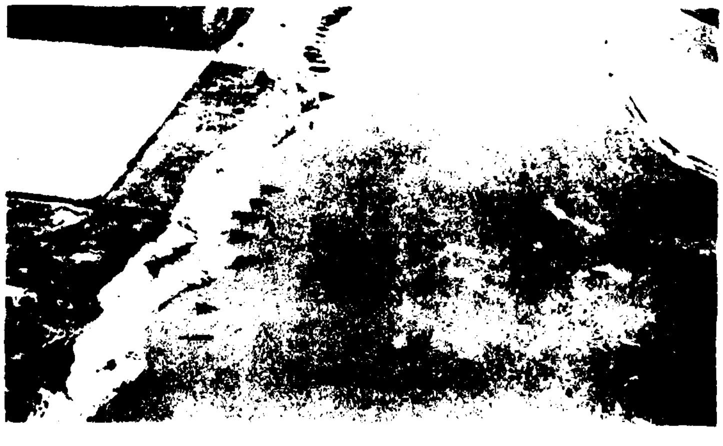
PROJECT LAYOUT AND TYPICAL SECTION

ITEM 5-2

DURING CONSTRUCTION



AFTER CONSTRUCTION



AFTER CONSTRUCTION

RED RIVER AT PEROT, LA.

ITEM 5-3

BIG CREEK
BIG CREEK, LOUISIANA

Streambank Erosion Control Evaluation and Demonstration Act of 1974
Section 32 Program - Work Unit 2

EVALUATION OF EXISTING BANK PROTECTION WORKS

(1) Location

Stream Big Creek River Mile 7.0 Side Left and Right

Local Vicinity Big Creek, LA Lat W91°37'30" Long N32° 26' 00"

At/Nr City Holly Ridge Parish Richland, Franklin State LA Cong Dist 5

CE Office Symbol LMVD Responsible Agency Corps of Engineers

Site Map Sources ______

Land Use Information Sources ______

(2) Hydrology at or Near Site

Stage Range ____ to ____ ft; Period of Record 19__ to 19__.

Discharge Range 0 to 10,000 cfs; Velocity Range 0 to 4.0 fps

Sediment Range ____ to ____ tpd; Period of Record 19__ to 19__.

Bank-full Stage ____ ft; Flow ____ cfs; Average Recurrence Interval ____ yr

Bank-full Flow Velocity: Average ____ fps; Near Bank ____ fps

Comments ______

(3) Geology and Soil Properties

Bank (USCS) silt and clay lenses Bed (USCS) poorly graded, gravelly sands (SP)

Data Sources ______

Groundwater Bank Seepage ______

Overbank Drainage ______

Comments Upper 8-10 feet of bank sandy clay (CL) and fat clay (CH); next 10 feet poorly graded, gravelly sands (SP)

(4) Construction of Protection

Need for Protection ______

Erosion Causative Agents Vertical channel realignment resulted in a steeper bed slope and higher flow velocities; grade control was necessary to prevent bed degradation and possible bank failure.

Protection Techniques Sheet pile weir with stone paving upstream and downstream of pilings.

General Design Weirs installed to maintain minimum pool with negligible effect on flood flows.

Project Length ____ ft; Construction Cost $ 3.5 million for 4 weirs Mo/Yr Completed 7/77

(5) Maintenance

Experienced Flows (Stage, cfs, Date) ______

Repairs and Costs (Item, Cost, Data) Considerable riprap failure on side slopes; $77,000 to date.

Comments: Sheet pile weirs with riprap paving show economic potential for grade control; initial cost favorable. Repair cost unfavorable.

(6) Performance Observations and Summary

Monitoring Program ______

Documentation Sources ______

Project Effect on Stream Regime Minimum water levels being maintained reducing in-channel vegetation.

Project Effect on Environment ______

Successful Aspects ______

Unsuccessful Aspects ______

General Evaluation Unprotected energy hole allows downstream scour.

Recommendations ______

(7) Additional Information, Comments, and Summary

Map No. 6.

Attached Items.

6 - 1 - Project Summary

6 - 2 - Vicinity map

6 - 3 - Typical low-water weir

6 - 4 - Photographs after construction

6 - 5 - Photographs 3½ yrs after construction

BIG CREEK, LOUISIANA

The Vicksburg District Corps of Engineers constructed four drop structures (weirs) on Big Creek near where it empties into Boeuf River (Plate 1). The construction site is about 10 miles West of Winnsboro, Louisiana. The structures were broad-crested type weirs (Plate 2) constructed with sheet piling and riprap for low water control in a gravelly sand subjected to groundwater seepage, surface runoff, and stream currents with drift turbulence. One of the four weirs required repairs for upstream sloughing (Plate 3) and downstream bank erosion near the energy scour hole (Plate 4). The weirs were monitored for adequacy of design and to determine their effectiveness in controlling the grade of the creek. Two publications are available on this project.

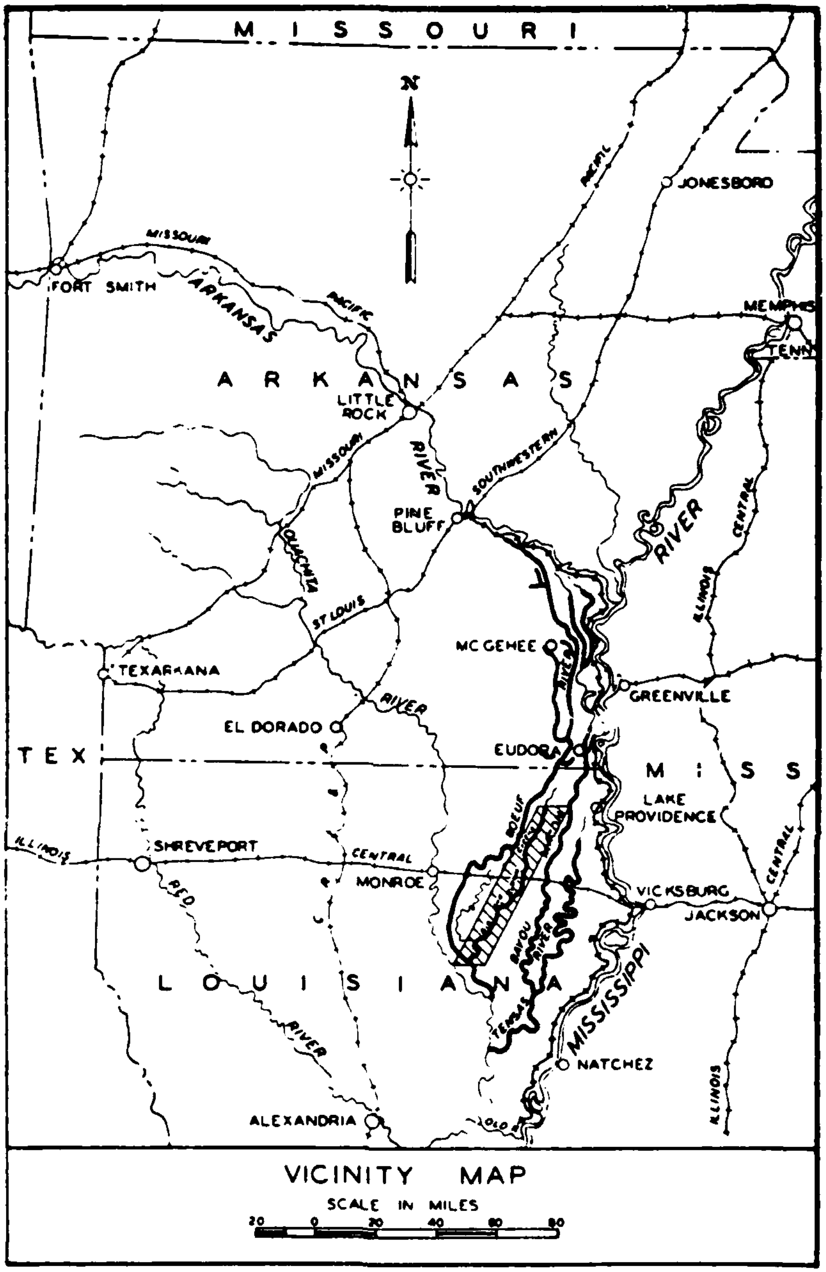
Ables, J. H., Jr., and Boyd, M. D. 1969 (Oct). "Low-Water Weirs on Boeuf and Tensas Rivers, Bayou Macon, and Big and Colewa Creeks, Arkansas and Louisiana; Hydraulic Model Investigation," Technical Report H-69-13, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, MS.

Miller, S. P. 1978 (Feb). "Bank Distress of Low-Water Weirs on Big Creek, La.," Miscellaneous Paper S-78-2, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, MS.

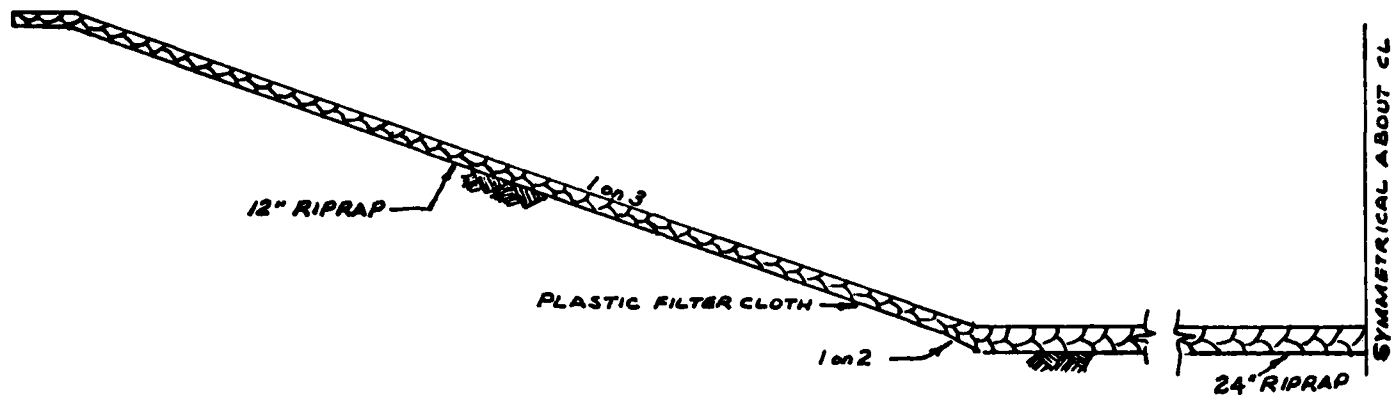
*Banks (USCS) Upper 8-10 feet of bank sandy clay (CL) and fat clay (CH); next 10 feet poorly graded, gravelly sands (SP) with silt and clay lenses; below 10 feet is generally poorly graded, gravelly sands (SP).

PROJECT SUMMARY

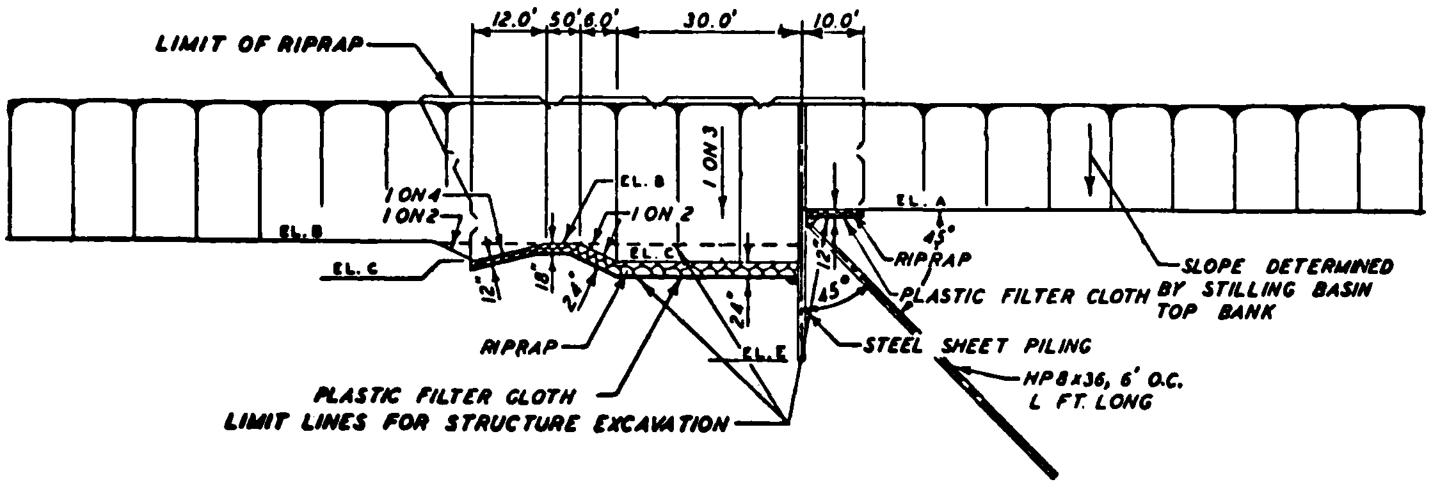
ITEM 6-1


MISSOURI
JONESBORO
MISSOURI
FORT SMITH
ARKANSAS
PACIFIC
MEMPHIS
TENN
ARKANSAS
LITTLE ROCK
RIVER
SOUTHWESTERN
PINE BLUFF
RIVER
CENTRAL
OUACHITA
ST LOUIS
ILLINOIS
MC GEHEE
TEXARKANA
GREENVILLE
EL DORADO
RIVER
EUDORA
TEX
MISS
LAKE PROVIDENCE
BOEUF
ILLINOIS
SHREVEPORT
CENTRAL
MONROE
RED
VICKSBURG
JACKSON
CENTRAL
LOUISIANA
BAYOU
TENSAS
MISSISSIPPI
RIVER
NATCHEZ
ILLINOIS
ALEXANDRIA
OLD R
VICINITY MAP
SCALE IN MILES
20 0 20 40 60 80

ITEM 6-2



HALF-SECTION



CENTER LINE PROFILE

Typical weir cross sections

BIG CREEK, LA.

TYPICAL LOW-WATER WEIR

AFTER CONSTRUCTION MAY 1977
Weir 4 Upstream Bank Sloughing

3 YEARS AFTER CONSTRUCTION JULY 1980
Weir 4 Downstream

BIG CREEK, LA.
PHOTOGRAPHS IMMEDIATELY AFTER AND 3-1/2 YEARS AFTER
CONSTRUCTION OF WEIR 4

ITEM 6-4

GENERAL VIEW OF WEIR

ERODED BANK DOWNSTREAM OF WEIR DUE TO OVERBANK RUNOFFS

BIG CREEK, LA.
PHOTOGRAPHS OF WEIR 4, 3-1/2 YEARS AFTER CONSTRUCTION

ITEM 6-5

ST. CATHERINE CREEK
NATCHEZ, MISSISSIPPI

Streambank Erosion Control Evaluation and Demonstration Act of 1974
Section 32 Program - Work Unit 2

EVALUATION OF EXISTING BANK PROTECTION WORKS

(1) Location

Stream St. Catherine Creek River Mile 2.0 Side Right

Local Vicinity Natchez Lat N 30°31'12" Long W 91°25'48"

At/Nr City Natchez County Adams State MS Cong Dist 4

CE Office Symbol LMVD Responsible Agency Corps of Engineers

Site Map Sources ______

Land Use Residential

(2) Hydrology at or Near Site

Stage Range ____ to ____ ft; Period of Record 19__ to 19__.

Discharge Range 31,000 to ND cfs; Velocity Range 5.07ND to 8.95MP fps

Sediment Range ____ to ____ tpd; Period of Record 19__ to 19__.

Bank-full Stage ____ ft; Flow ____ cfs; Average Recurrence Interval ____ yr

Bank-full Flow Velocity: Average ____ fps; Near Bank ____ fps

Comments ______

(3) Geology and Soil Properties

Bank (USCS) Very fine silt (loess) Bed (USCS) Sand and gravel deposits

Data Sources ______

Groundwater Bank Seepage ______

Overbank Drainage ______

Comments ______

(4) Construction of Protection

Need for Protection Storm events were endangering residential area

Erosion Causative Agents High stage streamflow against concave bank of bendway

Protection Techniques Tire revetment placed by local interests.

General Design ______

Project Length 300 ft; Construction Cost $ 1,000 Mo/Yr Completed 1973

(5) Maintenance

Experienced Flows (Stage, cfs, Date) ____________________

Repairs and Costs (Item, Cost, Data) ____________________

Comments: None to date.

(6) Performance Observations and Summary

Monitoring Program Onsite inspection.

Documentation Sources ____________________

Project Effect on Stream Regime ____________________

Project Effect on Environment Helpful-vegetation established with a more stable bank.

Successful Aspects Has retarded erosion.

Unsuccessful Aspects Upstream erosion may threaten future performance of streambank protection.

General Evaluation Dr. Tillman (private landowner who did the work) thinks this approach is ideal for private landowners to stabilize streambank with loess soil.

Recommendations Good technique for private landowners.

(7) Additional Information, Comments, and Summary

Map No. 7.

Attached Items:

7 - 1 - Project summary

7 - 2 - Photographs 8 yrs after construction

ST. CATHERINE CREEK AT NATCHEZ, MISSISSIPPI

Dr. Clifford Tillman, Natchez, MS, owns property on St. Catherine Creek which includes a 350-ft section adjacent to the stream. The bank is approximately 22 ft high along Dr. Tillman's property; however, the bank rises to 40 ft upstream and downstream. Prior to 1972, Dr. Tillman constructed a tire mattress consisting of some 2000 factory-reject tires and 15,000 ft of steel cable. Truck tires were used at the toe of the revetment and automobile tires above the toe. Holes were drilled through the tread wall of each tire; between 12 and 15 tires were then strung on a 1/2-in. steel cable. After connecting the cable to a steel pipe set 50 ft back from top bank, the string was placed vertically down the bank. After the vertical strings of tires were in place, they were woven together horizontally, using 1/2-in. cable for the automobile sides and toe of the mattress by folding the cable back over itself around the tire and then attaching cable clamps. The upstream end of the row of truck tires was anchored to a 2-ton steel beam; the mattress was not anchored at any other location along the sides or toe. Dr. Tillman also placed 32 cypress pilings at the upstream end of the revetment to protect the mattress. The bank was not shaped prior to placement of the revetment; however, the mattress adjusted well to the bank geometry. The revetment construction began in 1972 and was completed in 1973. Dr. Tillman and his teenaged son provided most of the labor; however, Dr. Tillman hesitated to make any estimate of the time required to complete the work. The total expenditure was less than $1000, with the tires, cable, and steel beam being donated. Only 280 ft of the 300-ft-long used tire revetment (constructed in 1972 by weaving steel cable through the tires) has been successful at retarding erosion. A storm event in 1979 removed the upstream 20 ft of the revetment; however, the remainder of the revetment has remained intact with vegetation establishing itself on the surface of the revetment. Dr. Tillman is pleased with the performance of the mattress; however, he is concerned that the project will be lost in the future because property owners upstream and downstream have not been able to protect their banks. The 32 cypress pilings placed at the

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(Sheet 1 of 2)

upstream end of the mattress have been lost; Dr. Tillman feels they were not driven deep enough. In addition, the upstream 10-15 vertical strings of tires have also failed. Willows and cottonwood are now well established over the remaining surface area of the revetment.

Based on the performance of the mattress thus far, Dr. Tillman thinks this approach is ideal for the private landowner who is attempting to stabilize streambanks with loess-type soil.

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(Sheet 2 of 2)

LITTLE BLUE RIVER
INDEPENDENCE, MISSOURI

Streambank Erosion Control Evaluation and Demonstration Act of 1974
Section 32 Program - Work Unit 2

EVALUATION OF EXISTING BANK PROTECTION WORKS

(1) Location

Stream Little Blue River River Mile 7.4-11.5 Side Both

Local Vicinity ______ Lat N39°06' Long W94°18'

At/Nr City Independence County Jackson State MO Cong Dist 4

CE Office Symbol MRK Responsible Agency Corps of Engineers

Site Map Sources Corps of Engineers, Kansas City District

Land Use Homes, farming, and industrial

(2) Hydrology at or Near Site

Stage Range 0 to 23 ft; Period of Record 1948 to 1981.

Discharge Range 0 to 17,000 cfs; Velocity Range 0 to 9 fps

Sediment Range 0 to 253,144 tpd; Period of Record 1971 to 1981.

Bank-full Stage 23.5 1/ ft; Flow ______ cfs; Average Recurrence Interval 100 yr

Bank-full Flow Velocity: Average 9 fps; Near Bank ______ fps

Comments Bed gradient 3 ft/mile. 1/ Improved channel.

(3) Geology and Soil Properties

Bank (USCS) Sandy silts to lean clays Bed (USCS) Sandy, silty clay

Data Sources Corps of Engineers test borings.

Groundwater Bank Seepage NA

Overbank Drainage ______

Comments ______

(4) Construction of Protection

Need for Protection Highly erodible soils in 12 areas of improved channel

Erosion Causative Agents Erodible soils in areas of turbulence and channel alignment.

Protection Techniques Riprap of side slopes of low flow channels with short horizontal blanket at toe.

General Design 18 - 21 in riprap on 6 in. layer of bedding on 1V-2H slope

Project Length ______ ft; Construction Cost $ * Mo/Yr Completed 12/78

*Construction cost cannot be isolated from overall project costs.

(5) Maintenancc

Experienced Flows (Stage, cfs, Date) 17,000 cfs Sept. 1977

Repairs and Costs (Item, Cost, Data) None to date.

Comments:

(6) Performance Observations and Summary

Monitoring Program Visual inspections, cross-sectioned December 1980.

Documentation Sources

Project Effect on Stream Regime

Project Effect on Environment Nothing adverse.

Successful Aspects Maintaining efficient channel. Horizontal blanket at toe effectively preventing undercutting of slope riprap.

Unsuccessful Aspects None apparent.

General Evaluation Very effective erosion control.

Recommendations

(7) Additional Information, Comments, and Summary

Map No. 8.

Attached Items:

8-1 Project summary and location

8-2 Project cross section and photograph

Little Blue River at Independence, MO. (Mile 7.4 to 11.5)

The Little Blue River is a right bank tributary of the Missouri River, joining the main stem at mile 339.5 (1960 adjustment) 20 miles downstream from Kansas City, Mo. The Little Blue Basin is 33 miles long, with a maximum width of 13 miles. The total drainage area of the basin is 224 square miles of which 90 percent is in Jackson County, Mo., with the remainder being in Cass County, Mo. The lower 7.4 miles of the Little Blue River are confined to an improved channel between the right bank bluffs of the Missouri River floodplain and the Unit R-351 tieback levee of Missouri River Levee System.

The Little Blue River Basin is frequently subject to flooding from high-intensity rainstorms mostly during the months of April through October. Flood stage at the Lake City gage has been exceeded 21 of the 23 years since records have been kept. The gradual encroachment of the Kansas City metropolitan complex into the basin has significantly raised the flood damage potential. To mitigate this threat, a project for channel improvement and reservoir construction was authorized for the Little Blue Basin by the 1968 Flood Control Act, Public Law 90-483 (part of the comprehensive plan for the Missouri River Basin). This legislation provides for channel improvement (in four stages) from mile 7.4 through mile 22.4, and the construction of Longview Dam upstream from the Main-stem channel improvements and Blue Springs Lake Dam on the East Fork of the Little Blue River. The channel improvement feature incorporated two types of channels: a low-flow channel that follows much of the bed of the existing Little Blue River, and a high-flow channel 5 ft in elevation above the existing waterway to handle flood discharges. When the project is completed, the natural channel length will be shortened from 22 miles to 15 miles for the high-flow channel and to 18 miles for the low-flow channel.

ITEM 8-1
(Sheet 1 of 4)

The Stage I channel improvement contract was awarded on 20 December 1975, Figure 1, and was completed on 5 December 1978. Three Section 32 existing sites were selected in the Stage I reach. They were:

a. Stone riprap on the side slopes of the low-flow channel.

b. Sheet piling and rock sills.

c. Compacted clay on the berm and side slopes of the high-flow channel.

The structures placed at each of these sites were designed to withstand a 100-year flood. This report is concerned with the low-flow channel riprap.

Since March 1948, the U.S. Geological Survey (USGS) has maintained a stream gaging station at the Missouri State Highway 78 bridge (reported as the Lake City gage). The daily discharges of record are: maximum 17,000 c.f.s. (during the September 1977 flood); mean 133 c.f.s., and minimum no flow on several occasions. The improved channel through Stage I is designed for 18,000 c.f.s. (100-year flood). The maximum observed stream velocity is 9 ft/sec which occurred during the 1977 flood. A suspended-sediment sample station has been operated at this site since October 1971. The maximum load for the period of record is 253,144 tons/day, the mean 426 tons/day, and the minimum 0 tons/day. The maximum annual load of record is 374,933 tons (water year 1977); the average annual load is 155,556 tons. Average sediment load in this reach consists of 6 percent sand, 51 percent silt, and 43 percent clay. The average annual sediment yield upstream from the Stage I reach is 750 to 1,000 tons per square mile. Soil types vary from clays in the upper end of the Stage I reach (CL and CH) to sands at the lower end (SM, SP, and sandy ML).

The side slopes of the low-flow channel were protected by an 18-in.-thick layer of Type A stone riprap placed over a 6-in.-thick layer of bedding material, Figures 2 and 3, in reaches where a minimum of

ITEM 8-1
(Sheet 2 of 4)

protection was required. In high-velocity or turbulent environments (bends, downstream from structures, etc.), a 21-in.-thick layer of Type B riprap was placed over the bedding material and a short horizontal toe section was placed at the toe of the side slope riprap. Riprap was required at 12 locations through the Stage I reach, Figure 1, to stabilize the low-flow channel. All materials were brought onsite by truck; an orange peel bucket and crane was then used for below water placement, and a Gradall for above water placement. The Type A and Type B stone riprap was specified to meet the following gradations:

Weight per Stone lb	Percent of Total Weight Lighter Than
Type A	
250	100
180	85-95
60	30-50
10	0-10
Type B	
600	100
450	85-95
150	30-50
20	0-10

The riprap was required to be approximately rectangular in cross section, to be relatively free from slabby pieces and deleterious substances, and to have an elongation ratio not exceeding 3. The bedding material was specified to meet the following gradation:

Sieve Size	Percent by Weight Passing
4 in.	100
3 in.	75-95
3/4 in.	40-60
3/8 in.	20-40
No. 4	5-25

Material not passing the 3/4-in. sieve was specified to be reasonably free from flat elongated particles and deleterious substances.

The flood of record in September 1977 (17,000 c.f.s.) caused no damage to any of the structures in the Stage I reach. Visual inspections of the Stage I reach through January 1981 indicate that the bank protection measures are performing well.

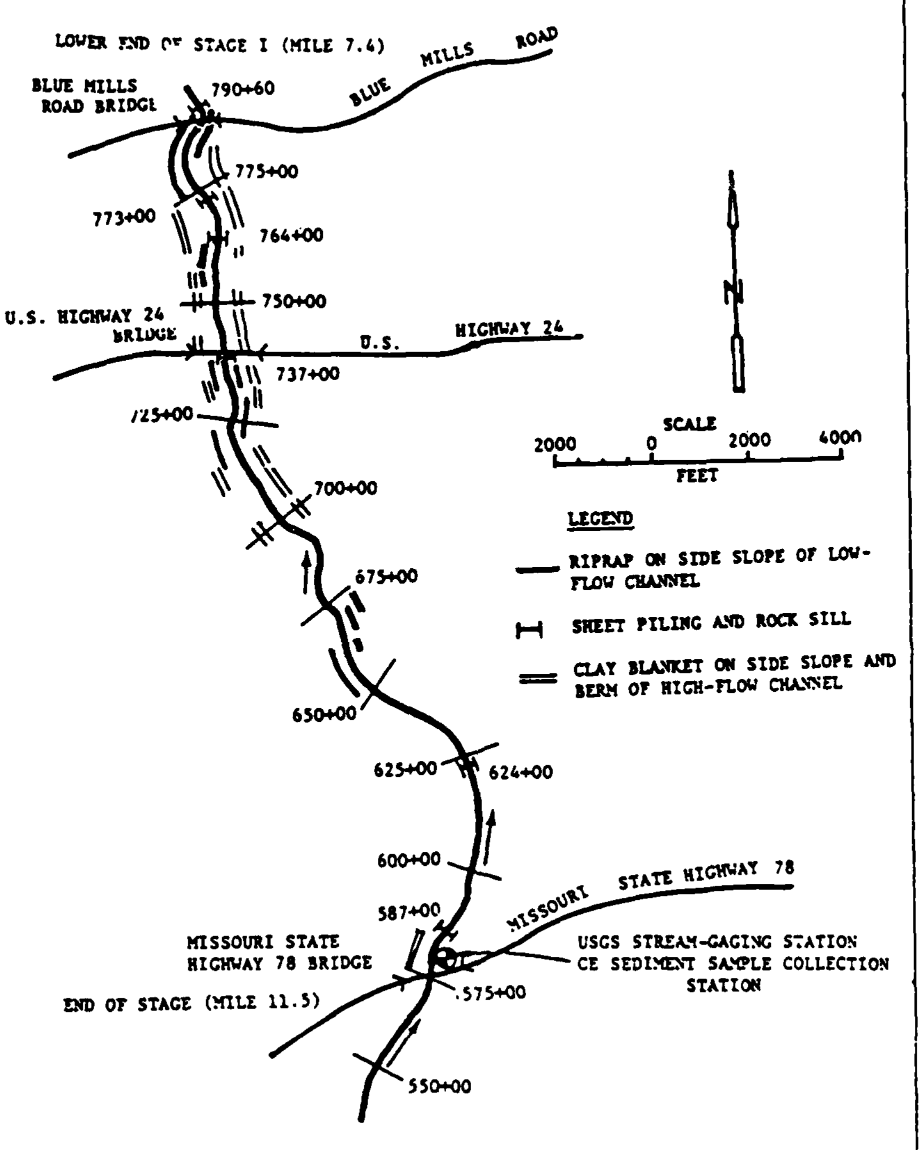


Figure 1. Little Blue River channel improvement at Independence, Mo., Stage I (sta 577+00 to 788+96)

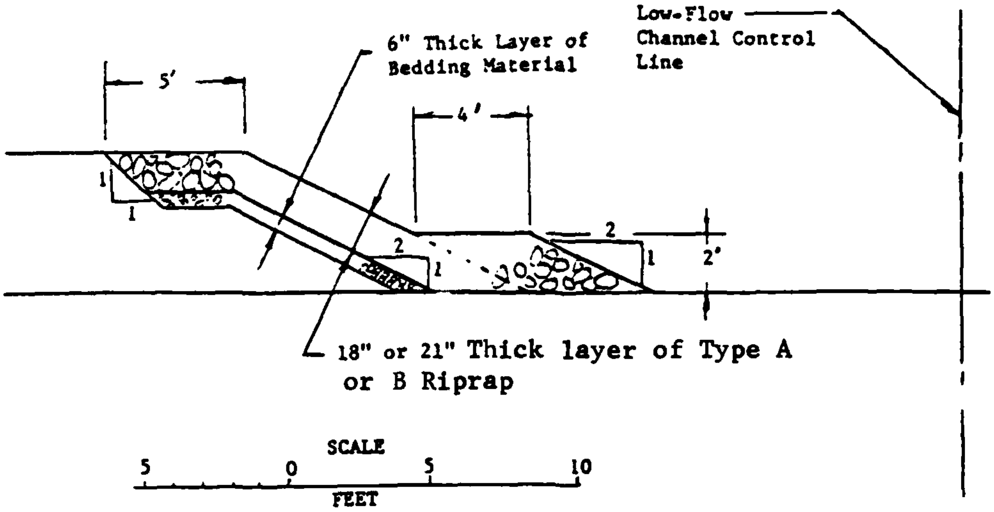


Figure 2. Little Blue River at Independence, Mo. Cross-sectional view of low-flow channel riprap side-slope revetment.

Figure 3. Little Blue River at Independence, Mo. Side slopes of low-flow channel protected by stone riprap

ITEM 8-2

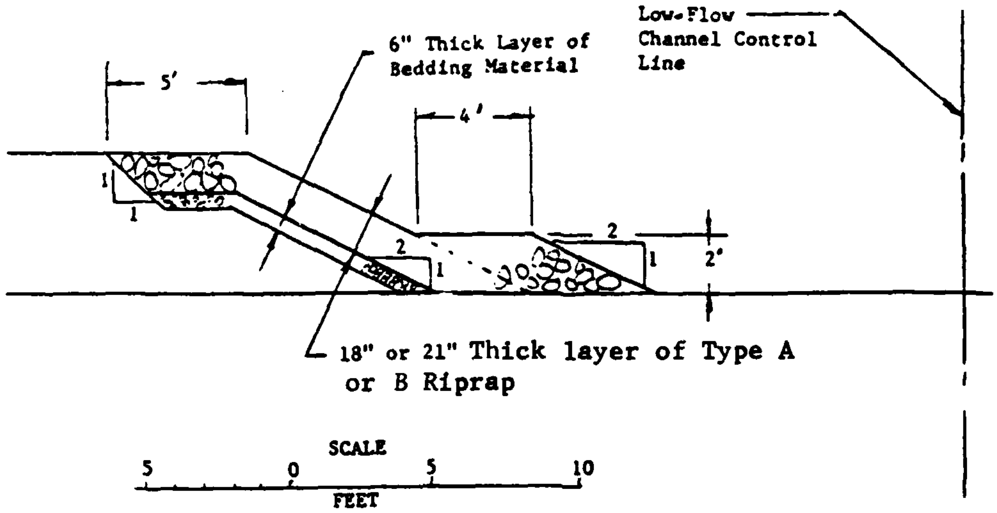


Figure 2. Little Blue River at Independence, Mo. Cross-sectional view of low-flow channel riprap side-slope revetment.

Figure 3. Little Blue River at Independence, Mo. Side slopes of low-flow channel protected by stone riprap

ITEM 8-2

REPUBLICAN RIVER
MILFORD DAM, KANSAS

Streambank Erosion Control Evaluation and Demonstration Act of 1974
Section 32 Program - Work Unit 2

EVALUATION OF EXISTING BANK PROTECTION WORKS

(1) Location

Stream Republican River River Mile 6.1-7.3 Side Both

Local Vicinity ______ Lat N39°04' Long W96°52'

At/Nr City Junction City County Geary State KS Cong Dist 2

CE Office Symbol MRK Responsible Agency Corps of Engineers

Site Map Sources Corps of Engineers, Kansas City District

Land Use Recreation area.

(2) Hydrology at or Near Site

Stage Range ______ to ______ ft; Period of Record 1967 to 1981.

Discharge Range 15 to 12,500 cfs; Velocity Range ______ to ______ fps

Sediment Range 0.8 to 11,037 tpd; Period of Record 1967 to 1974.

Bank-full Stage ______ ft; Flow 23,000 cfs; Average Recurrence Interval 100 yr

Bank-full Flow Velocity: Average 5 fps; Near Bank ______ fps

Comments Original channel designed for 15,000 cfs at 9.3 fps.

(3) Geology and Soil Properties

Bank (USCS) Fine silts and sands Bed (USCS) Fine silts and sands

Data Sources Corps of Engineers test borings.

Groundwater Bank Seepage ______

Overbank Drainage ______

Comments ______

(4) Construction of Protection

Need for Protection Outlet channel from Milford Dam was rapidly widening due to lateral erosion.

Erosion Causative Agents Channel degradation downstream from reservoir.

Protection Techniques Rock revetment with horizontal toe blanket.

General Design Rock revetment placed on 1V on 3H slope with 3-ft.-thick blanket of stone placed horizontally at the base of the side slope (See page H-9-9 for comments).

Project Length 7,000 ft; Construction Cost $ 400,000 Mo/Yr Completed 1/69

(5) Maintenance

Experienced Flows (Stage, cfs, Date) 500 cfs at least 50% of time from 1/69 to 12/74. In 1973 flow of 8,000-12,000 cfs for 44 days.

Repairs and Costs (Item, Cost, Data)

Comments:

(6) Performance Observations and Summary

Monitoring Program Periodic inspections, surveys, and check of gradation.

Documentation Sources MRK

Project Effect on Stream Regime Stabilized banks.

Project Effect on Environment

Successful Aspects As the channel degraded, stone from toe blanket migrated down and revetted the slope.

Unsuccessful Aspects Some separation in protective blanket at the point of original toe elevation.

General Evaluation Banks stabilized at appr. one-half the cost of riprap placement to expected depth of degradation.

Recommendations Horizontal toe blanket should contain 1-1/2 times the amount of stone required to continue riprap protection to anticipated depth of channel degradation.

(7) Additional Information, Comments, and Summary

Map No. 9. Four special test sections at downstream end of outlet channel, two horizontal blankets and two sections with enlarged section at base of slope.

Attached Items:

9-1 Project summary and location 9-(4-8) Photographs
9-2 Cross sections
9-3 "

Republican River at Milford Dam Outlet Channel, Kansas (Mile 6.1 to 7.3)

The 1954 Flood Control Act (Public Law 83-780) authorized construction of Milford Dam at mile 7.7 on the Republican River (4 miles northwest of Junction City, Kansas) as a part of the comprehensive plan for flood control in the Missouri River Basin (Figure 1). Milford Dam is a compacted earth and rock-fill embankment with an impervious core, being 6,300 ft long and constructed to an elevation of 1,213 ft (126 ft above the valley floor). Multipurpose reservoir operations began on 16 January 1967. The design storage capacity of the reservoir is 1,160,000 acre-ft, which includes 700,000 acre-ft allocated for flood control, with the remainder being for multipurpose use.

The USGS has operated a gaging station on the Republican River at mile 6.0 since 1 October 1963. Daily discharges of record prior to the date the dam became operational (1 October 1963-16 January 1967) were: minimum 17,200 c.f.s., mean 675 c.f.s., and minimum 9.0 c.f.s. The daily discharges of record after the dam became operational (16 January 1967-present) are: maximum 12,600 c.f.s., mean 839 c.f.s., and minimum 15 c.f.s. No suspended-sediment samples were obtained in the outlet channel reach prior to closure of Milford Dam; however, a suspended-sediment sample collection station was operated at mile 6.0 from 1 October 1967 through 30 September 1974. The daily suspended sediment loads of record were: maximum 11,037 tons, mean 84 tons, and minimum 0.81 ton. The maximum annual suspended-sediment load was 111,172 tons (water year 1974); the average annual load was 31,875 tons.

The 8,000-ft-long Milford Dam Outlet Channel was originally excavated through highly erodible fine silts and sands to a 100-ft bottom width with 1V-on-3H side slopes. Riprap was placed on the side slopes for a distance of approximately 1,000 ft downstream of the stilling basin. Within a short period of time the remaining 7,000-ft length of this channel had eroded to

ITEM 9-1
(Sheet 1 of 8)

a width of approximately 200 ft and was threatening to encroach into a proposed recreation area. To prevent this encroachment, it was proposed to retain the existing 200-ft bottom width, grade the banks to a 1V-on-3H side slope, and then pave the banks with a 12-in.-thick layer of rock having a median weight of 25 lb and a maximum weight of 150 lb. The stone was to be placed over a 6-in.-thick filter blanket in order to prevent loss of the fine-grained bank material through the riprap. Seven to 10 ft of degradation was anticipated after the banks had been stabilized. Extending the side-slope revetment to that depth would have required up to 10 ft of underwater excavation. Past experience with these cohesionless soils had shown that 2 to 3 ft was the maximum working depth for excavation without dewatering when placing a controlled thickness of rock. The estimated cost at 1967 price levels of dewatering and placing the rock to the expected depth of degradation was approximately $900,000.

In view of this very high cost, it was proposed to place a horizontal blanket of rock on the streambed at the base of the side slopes, and thus as the bed degraded and undermined the blanket, the toe would be armored by the downward migration of stone placed as part of the horizontal blanket. The volume of rock in the blanket was initially estimated to be two times the volume required if the slope protection were extended to the expected depth of degradation. The estimated cost using the rock blanket approach was slightly over $400,000 or less than half the cost of extending the slope protection. Since the amount of rock actually required to provide sufficient protection as the bed degraded was not known, model testing was needed to see if additional savings might be realized.

Model testing was conducted by the Mead Hydraulics Laboratory located at the University of Nebraska Field Station near Mead, Nebr. (a facility jointly used by the University of Nebraska and MRD). Ten different toe geometries were tested. The model tests confirmed that the horizontal blanket proposed as toe protection for the outlet channel revetment would

perform as expected when the bed degraded. They also confirmed that the volume of rock in the blanket could be substantially less than twice the volume of extending the revetment to the anticipated depth of degradation.

Placement of riprap from mile 6.1 to 7.3 (the Section 32 existing site) began in August 1966 and was completed in early 1969. Based on the results of the Mead Laboratory model tests, the horizontal blankets were placed 3 ft thick with the width varying from 12 to 17 ft. This technique provided a quantity of rock approximately 1-1/2 times the volume that would have been required to extend the slope protection to the expected depth of degradation. Specifications for the 12-in.- thick layer of stone riprap pavement on the side slopes were noted in previous paragraph. The requirements for the 6-in.-thick layer of bedding material beneath the side-slope pavement were:

Sieve Size	Percent by Weight Passing
4 in.	Maximum allowable
2 in.	75-95
1 in.	35-65
1/4 in.	5-30
No. 40	0-15

The completed revetment was designed to sustain a design discharge of 12,500 c.f.s. at a mean velocity of 5 fps. The bed gradient through the reach at the time of construction was essentially flat.

Four special toe test sections, two on each bank and each 200 ft long, were constructed at the downstream end of the outlet channel, Figure 1. The toe of Test Section 1 was constructed as an extension of the upper bank paving except that the lower 4 ft was placed on a 1V-on-1.5H slope, Figure 2. The base of the toe was 39 in. below the existing streambed at the time of construction. In Test Section 2, the toe was constructed as an enlarged section at the bottom of the revetment, Figure 3, with the base of the toe 16 in. below the streambed. Both the front and back of the toe section were placed on a 1V-on-1.5H slope. The break in slope between the toe section and the 1V-on-3H upper bank slope was

located 18 in. above the streambed. The toe of Test Section 3 was constructed as a 12-in.-thick horizontal blanket placed on the streambed, Figure 4. The original width of the blanket was estimated at 18 ft. Test Section 4 was placed on the streambed as a 2-1/2-ft-thick horizontal blanket of rock, Figure 5. The original width of the blanket was estimated at 10 ft.

Several periods of prolonged high releases from Milford Lake have occurred since placement of the revetment in the outlet channel. A discharge of 500 c.f.s. or more was passed at least 50 percent of the time between January 1969 and December 1974. For a period of 44 days in 1973, there were releases of 8,000 to 12,000 c.f.s. As a result of these flows, the bed of the outlet channel degraded an average of 5 to 6 ft through its entire length.

In December 1974, the outlet channel was inspected to evaluate the overall performance of the revetment toes. The inspection team found that the toes had performed their intended function well. The team probed the streambed at the base of the slope at several locations to determine the outer extremity of the riprap movement. In addition, they removed a small amount of rock to locate the original elevation of the base of the toe and to estimate the thickness of the stone layer formed by the downward movement of the riprap. They noted that the rock had moved downslope as the bed degraded until the bank was covered with a two-rock-diameter thickness. Approximately half of the toe stone was still in reserve at the original base of the revetment. The four special test sections were inspected in detail in order to compare the performance of the various toe geometries. Surveyed cross sections were obtained that extended from the top of the revetment to the bed and a short distance out into the channel for each test section, Figures 2-5.

Three to 3-1/2 ft of degradation had occurred below the original base of the toe of Test Section 1. The surveyed cross section indicated

ITEM 9-1
(Sheet 4 of 8)

that the entire riverward face of the toe had receded as the riprap moved downward and adjusted to the degradation, Figure 2. The toe rock had covered the lower slope quite well; however, there was some evidence of separation in the vicinity of the original base of the toe. Since there was little or no stone reserve remaining in this toe, the team concluded that progressive failure of the lower slope protection could occur if there were additional significant degradation.

Approximately 4-1/2 ft of degradation below the original base of the toe had occurred in Test Section 2, Figure 3. Riprap had covered the lower slope to a thickness of approximately two rock diameters. There appeared to be some evidence of separation along the original base elevation of the toe as a line of exposed bedding could be seen along the entire length of the test section.

The streambed in the vicinity of the toe of Test Section 3 had degraded approximately 6 ft, Figure 4. The riprap had moved downward to cover the lower slope; however, the thickness averaged somewhat less than one rock diameter, i.e., small areas of exposed sand could be seen over the entire face of the toe. There was a substantial reserve of stone remaining in the toe; however, the inspection team noted that if additional degradation occurred too rapidly, partial revetment failure could occur due to separation of the thin blanket. Alternatively, recession of the bank by leaching could cause additional riprap to move downward from the reserve and form a thicker bank covering.

Approximately 5 ft of degradation had occurred along the toe of Test Section 4, Figure 5. There was a uniform blanket of riprap on the lower slope averaging at least two rock diameters thick. No evidence of blanket separation or areas of exposed bed material were noted. The riverward edge of the stone was essentially at the base of the slope with very little migration of riprap out into the channel. A substantial reserve of rock that could provide material to accommodate additional degradation still remained.

In summary, the 1974 inspection indicated that the overall performance of the prototype test sections confirmed the results observed in the model tests. An extension of the slope protection showed evidence of stress soon after the base of the revetment was undercut (Test Section 1). The enlarged section at the base performed reasonably well; however, it appeared to have a tendency to eventually separate at the point where the steeper slope of the toe intersected the flatter slope of the upper bank paving (Test Section 2). A thin horizontal blanket did not release rock at a rate sufficient to provide an adequate thickness of coverage (Test Section 3). The performance to date of the thicker and narrower horizontal blanket (Test Section 4) was clearly superior to that of the other three test sections.

The model tests by Mead Laboratory indicated that only a single layer of riprap would form as the toe rock moved downward and that some channelward movement of stone would occur and form a horizontal apron at the base of the slope. The prototype performance of the thick horizontal blanket showed that a layer several rock diameters thick can develop under actual field conditions. There was also much less movement of riprap channelward at the base of the slope than was indicated by the model tests. Apron formation in the model may have been the result of the relatively large dune pattern in the model bed. In the model tests, the height of these dunes was approximately one-fourth of the water depth. At the time of the 1974 inspection, the streambed of the outlet channel was essentially flat, and even during high flows it was doubtful if dunes of significant height were formed. Final lower side slopes in the model were approximately 1V on 2H for all toe geometries. In the prototype, the slope below the horizontal toes (Test Sections 3 and 4) was approximately 1V on 2H; however, the slope extension (Test Section 1) and the enlarged base (Test Section 2) were somethat steeper below the original toe (about 1V on 1.5H). It was concluded that a volume of stone equal to 1-1/2 times the volume required to extend the slope protection to the expected depth of degradation provided an economic and efficient method

ITEM 9-1
(Sheet 6 of 8)

of protecting the revetment against damage by undercutting and was sufficient to withstand parallel flow conditions.

A team consisting of personnel from MRD and MRK conducted the 6th Periodic General Inspection of Milford Dam in May 1981. As part of this inspection, the outlet channel riprap was examined. The team found the rock protection on the channel side slopes was performing satisfactorily, Figure 6, even though the streambed had degraded nearly 6 ft. Progressive armoring was developing over the entire length of the channel bed, and the upstream one-half to two-thirds of the channel bed was armored to the point where no additional degradation was expected. No areas of weakness in the toe protection were noted during this inspection, Figure 7.

An inspection of the four toe test sections revealed some exposed areas between individual rocks. This was typical of the slope below the 12-in.-thick horizontal blanket in Test Section 3, Figure 8, and isolated areas over Test Sections 1 and 2, Figure 9. Good coverage of riprap on the slope below the 30-in.-thick horizontal blanket was noted in Test Section 4, Figure 10. No areas of exposed sand could be found in this test section. A substantial growth of willows, cottonwoods, and other woody vegetation had developed over Test Section 3, Figure 11. The team concluded that this growth could retard the downward movement of rock from this test section and that in order to retain a valid test, the vegetation should be removed. Breakdown of stone on the left bank near the waterline was noted. This was due to exposure to the sun during the winter and the resulting range of temperature variation that the riprap experiences; however, the extent of deterioration was not considered sufficient to affect the integrity of the revetment.

Figure 12 shows upstream and downstream views of the revetment in the outlet channel. The project is performing as designed, Figures 4 and 5. No apparent failures have occurred with the exception of minor areas where stone has been displaced and exposed the bedding material, Figures 8 and 9.

In August 1980, five test areas of stone were excavated and graded, Figures 13 and 14. The results showed the median size of the stone to be 22 pounds against the 25 pound median specified during construction and maximum size of 160 pounds against 150 pound maximum specified. This could indicate some breaking down of stone in the mid-range of the original gradation. The stone coverage in the test sections varied from an average of 1.5 to 2.25 rock diameters. Overall the revetment is performing as designed.

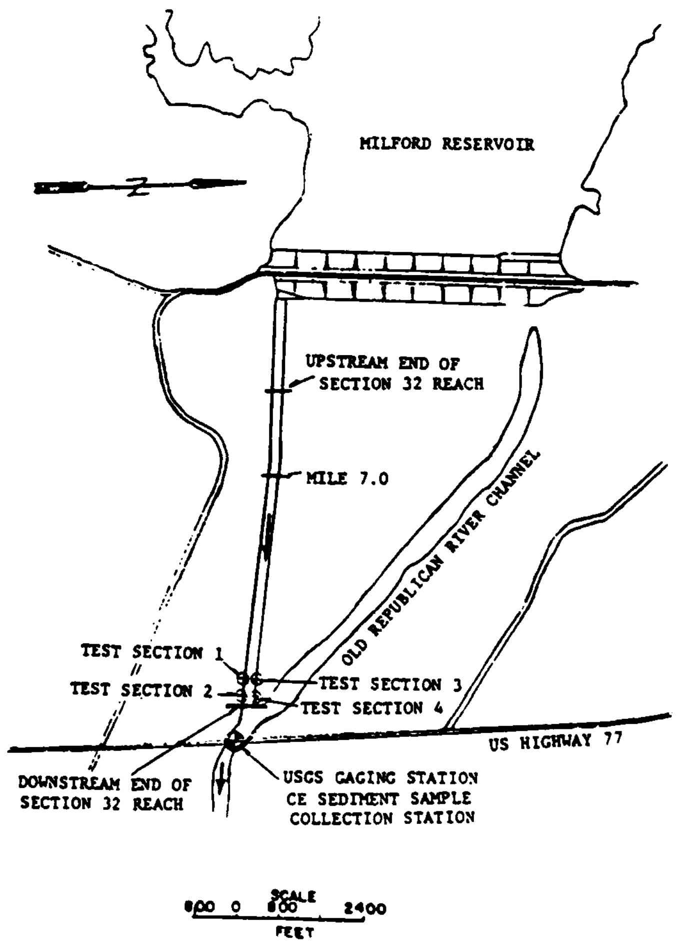


Figure 1. Republican River at Milford Dam Outlet Channel, Kans.

ITEM 9-1
(Sheet 8 of 8)

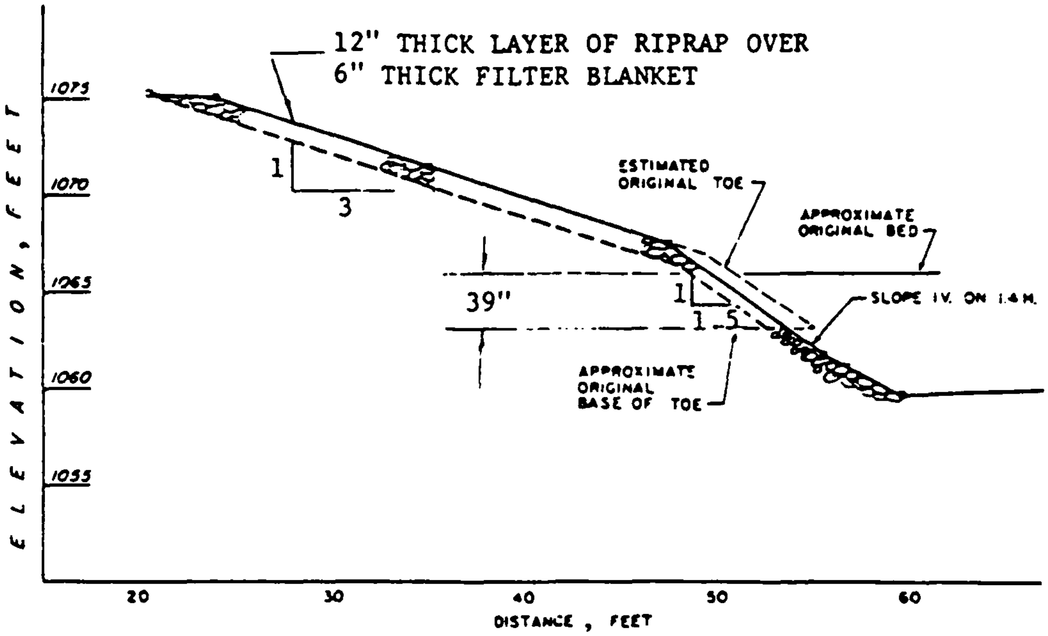


Figure 2. Republican River at Milford Dam Outlet Channel, Kans. Cross-sectional view of Test Section 1 shown as constructed in 1969 and as surveyed in 1974.

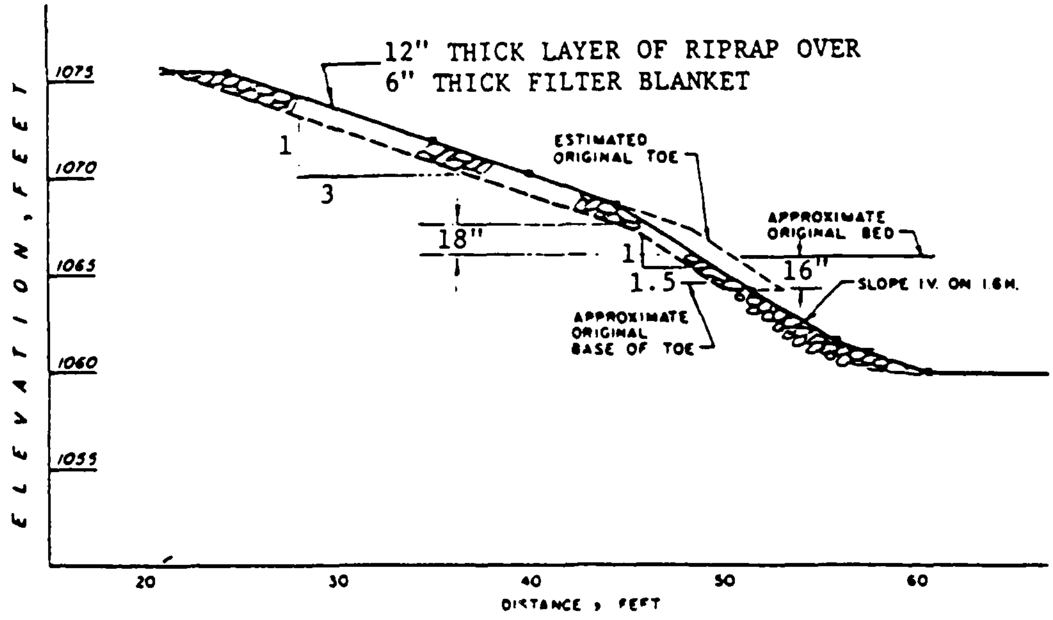


Figure 3. Republican River at Milford Dam Outlet Channel, Kans. Cross-sectional view of Test Section 2 shown as constructed in 1969 and as surveyed in 1974.

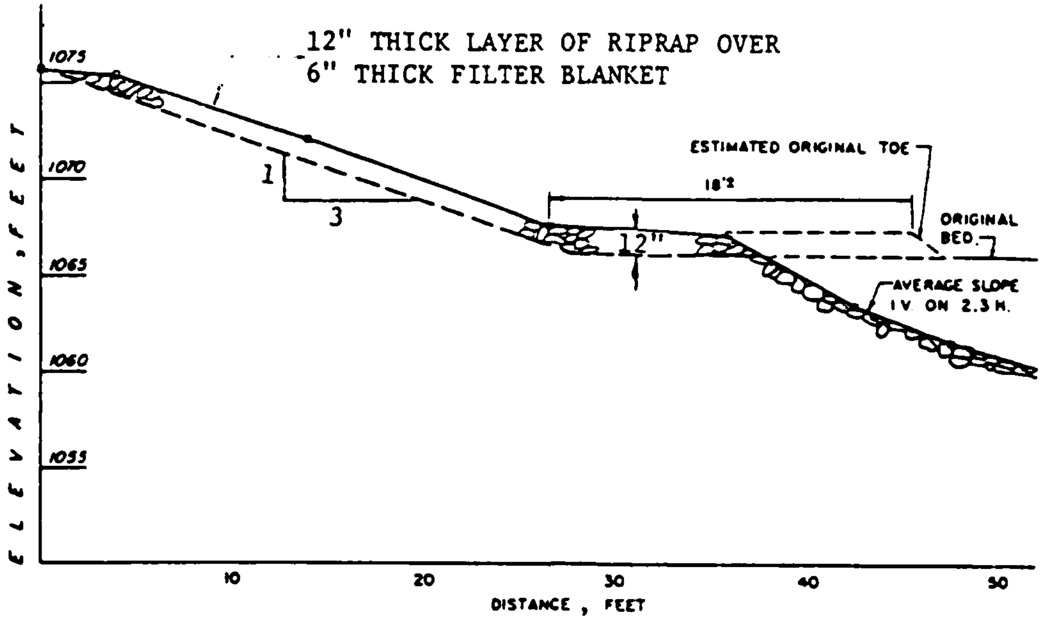


Figure 4. Republican River at Milford Dam Outlet Channel, Kans. Cross-sectional view of Test Section 3 shown as constructed in 1969 and as surveyed in 1974.

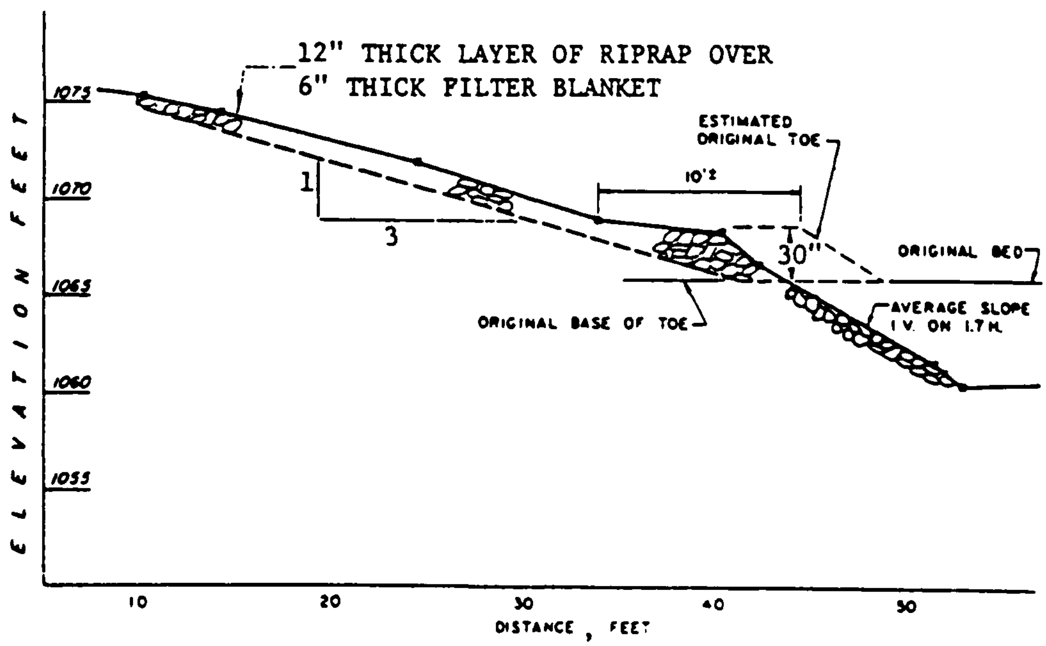


Figure 5. Republican River at Milford Dam Outlet Channel, Kans. Cross-sectional view of Test Section 4 shown as constructed in 1969 and as surveyed in 1974.

Figure 6. Republican River at Milford Dam Outlet Channel, Kans. General appearance of the middle reach of the outlet channel at the time of the July 1981 inspection.

Figure 7. Republican River at Milford Dam Outlet Channel, Kans. Typical view of the revetment toes as of July 1981, Note break in slope between the blanket and the slope to the water.

ITEM 9-4

Figure 8. Republican River at Milford Dam Outlet Channel, Kans. Areas of exposed sand visible between individual rocks below the 12-inch thick horizontal blanket in Test Section 3 (July 1981)

Figure 9. Republican River at Milford Dam Outlet Channel, Kans. The area is typical of the isolated areas of exposed bedding material in Test Sections 1 and 2 (July 1981).

ITEM 9-5

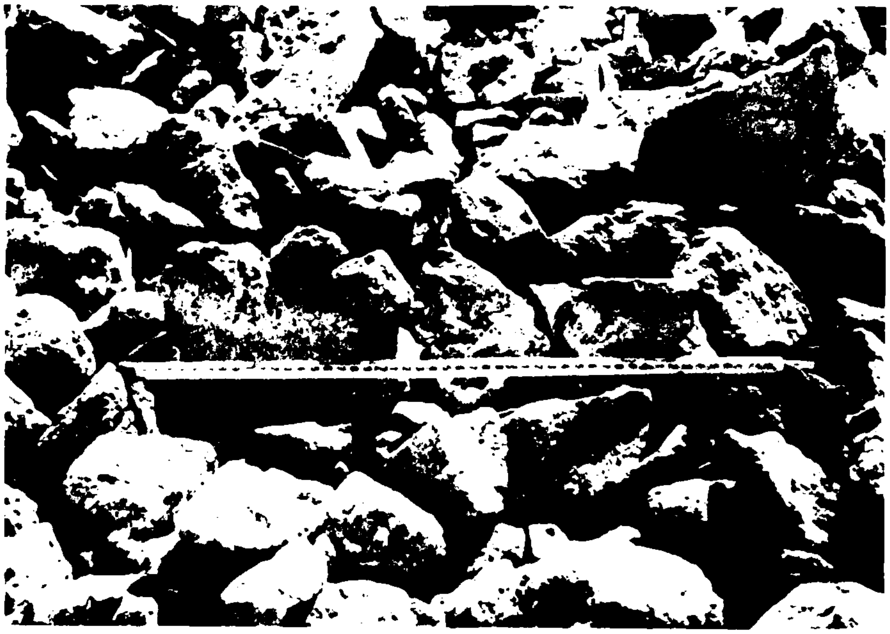


Figure 10. Republican River at Milford Dam Outlet Channel, Kans. There was good coverage of rock on the slope below the 30-in.-thick horizontal blanket at the time of the July 1981 inspection (Test Section 4).

Figure 11. Republican River at Milford Dam Outlet Channel, Kans. There was a substantial growth of woody vegetation over Test Section 3 at the time of the May 1977 inspection.

ITEM 9-6

a. Upstream

b. Downstream

Figure 12. Republican River at Milford Dam Outlet, Kans. Upstream and downstream views of the outlet channel revetment. (July 1981)

ITEM 9-7

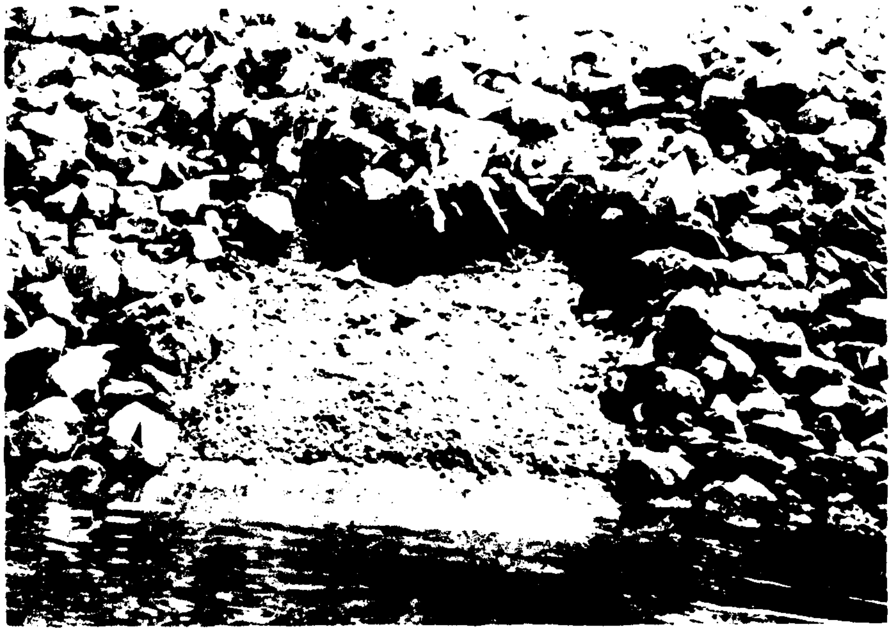


Figure 13. Republican River at Milford Dam Outlet, Kans. Test area sampled in August 1980 in Test Section 3.

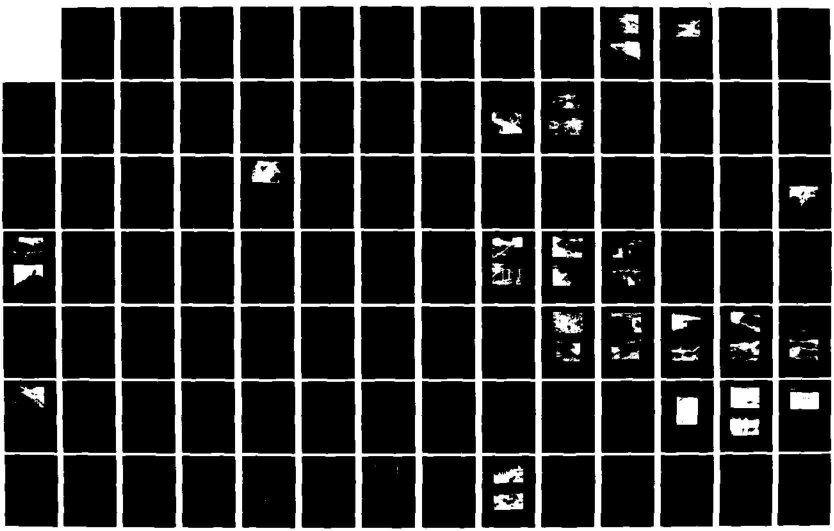
Figure 14. Republican River at Milford Dam Outlet, Kans. Test area sampled in August 1980 in Test Section 4.

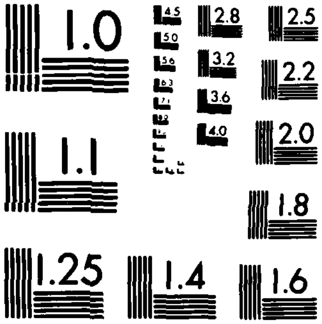
ITEM 9-8

LITTLE TIMBER CREEK
FRANKFORT, KANSAS

AD-A121 140 THE STREAMBANK EROSION CONTROL EVALUATION AND DEMONSTRATION ACT OF 1974 S..(U) ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG MS HYDRA..

UNCLASSIFIED M P KEOWN ET AL. DEC 81 F/G 13/2 NL





MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

Streambank Erosion Control Evaluation and Demonstration Act of 1974
Section 32 Program - Work Unit 2

EVALUATION OF EXISTING BANK PROTECTION WORKS

(1) Location

Stream Little Timber Creek River Mile ______ Side ______

Local Vicinity ______ Lat N39°42' Long W96°25'

At/Nr City Frankfort County Marshall State KS Cong Dist 2

CE Office Symbol MRK Responsible Agency Corps of Engineers

Site Map Sources Kansas City District, Corps of Engineers

Land Use Homes and light industry

(2) Hydrology at or Near Site

Stage Range NA to ______ ft; Period of Record 19__ to 19__.

Discharge Range NA to ______ cfs; Velocity Range ______ to ______ fps

Sediment Range NA to ______ tpd; Period of Record 19__ to 19__.

1/ Bank-full Stage ______ ft; Flow 2,200 cfs; Average Recurrence Interval ______ yr

1/ Bank-full Flow Velocity: Average 7 fps; Near Bank ______ fps

Comments 1/ Design computations made with grade-control structures in place.

(3) Geology and Soil Properties

Bank (USCS) Fat to lean clays (CH-CL) Bed (USCS) CH to CL

Data Sources Corps of Engineers test borings

Groundwater Bank Seepage ______

Overbank Drainage ______

Comments ______

(4) Construction of Protection

Need for Protection To prevent channel degradation and subsequent damage to adjacent levees.

Erosion Causative Agents Channel straightening causing steeper gradient and excessive velocities.

Protection Techniques Grade-control structures.

General Design Twin steel-sheet piling cutoffs with grouted stone between the pilings.

Project Length 1,500 ft; Construction Cost $ 2/ Mo/Yr Completed 1963

2/ Costs not separable from total project costs.

(5) Maintenance

Experienced Flows (Stage, cfs, Date)

Repairs and Costs (Item, Cost, Data) Damage to revetment below Check #1 has been repaired by sponsor.

Comments:

(6) Performance Observations and Summary

Monitoring Program Visual inspections.

Documentation Sources Kansas City District, Corps of Engineers

Project Effect on Stream Regime Stabilized channel.

Project Effect on Environment Negligible

Successful Aspects Prevented channel degradation.

Unsuccessful Aspects Large scour hole downstream due to lateral erosion.

General Evaluation

Recommendations Preformed scour hole should be riprapped.

(7) Additional Information, Comments, and Summary

Map No. 10.

Attached Items:

10-1 Project summary
10-2 Project location
10-3 Half plan
10-4 Half section
10-(5-6) Photographs

Tributary (Little Timber Creek) to the Black Vermillion River at Frankfort, Kansas (Mile 0.0 to 0.3)

Little Timber Creek is a right-bank tributary of the Black Vermillion River at Frankfort, Kansas. The Black Vermillion flows into the Big Blue River 11 miles downstream from Frankfort (28 miles upstream from Tuttle Creek Dam). Little Timber Creek drains a 13.5 square-mile watershed that is characterized by well-defined stream channels and gently rolling uplands. There are no long-term gaging or sediment records available for Little Timber Creek. The upstream average annual sediment yield is estimated to be 1,000 to 3,000 tons per square mile.

During the period 1903-1959, Frankfort experienced 18 damaging floods and numerous lesser overflows of streams surrounding the city. Prior to 1903, severe floods occurred, notably in 1844 and 1859, but reliable information on these early floods is not available. During the 30 May 1959 flood of record, the entire business district and about one-third of the residential area of Frankfort (235 acres) were inundated to depths that at some locations exceeded 6 ft. Between 1903 and 1960, at least five other floods reached elevations within 18 in. of the 1959 flood.* The severity of these flash floods depended chiefly on the timing of crests of Little Timber Creek, the Black Vermillion River, and West Fork (right-bank tributary of the Black Vermillion that enters the stream 2.1 miles downstream from the Little Timber-Black Vermillion confluence): thus all floods did not reach comparable depths in all parts of the floodplain.

The 1958 Flood Control Act (Public Law 85-500) authorized improvements in the vicinity of Frankfort which featured a 3.4-mile-long levee along the east, south, and west sides of the low-lying portion of the city, Figure 1. This levee had incorporated into its design 3 ft of freeboard

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* Data on the damaging floods were compiled from newspaper references to flood heights, areas inundated, relative depths of flooding, and from observed high-water marks.

ITEM 10-1
(Sheet 1 of 3)

above the design flood flow of 43,000 c.f.s. on the Black Vermillion. Other components of this project included channel improvements for 0.3 mile of Little Timber Creek and 1.5 miles of the Black Vermillion; a new bridge for the Missouri Pacific Railroad over Little Timber Creek; and gated drainage outlets through the levee. Construction was initiated on 9 March 1962 and the project was transferred to the city of Frankfort for operation and maintenance on 24 October 1963. The total Federal cost was $1,271,025 with the non-Federal cost estimated at $122,000.

Improvements on Little Timber Creek included the construction of a new channel beginning at the Fourth Street Bridge, Figure 1, and extending downstream to the mouth. Prior to construction, the thalweg of the existing channel dropped from el 1129.5 ft at the Fourth Street Bridge to el 1123.0 ft at the Missouri Pacific Railroad bridge, Figure 1, with an average bed slope of 10.5 ft/mile. Construction of a new channel following the route shown in Figure 1 would have increased the slope to 21.1 ft/mile; thus with the design discharge of 2,200 c.f.s. in the 14-ft bottom width channel and 1V-on-2H side slopes stream velocities approaching 10 fps would occur, corresponding to a Froude number of 0.73.

Experience on other projects in the Kansas City District had shown that new channels cut in soils characteristically similar to those found in the Frankfort area were subject to excessive bed or bank scour. This erosion often resulted in the undermining and failure of bank slopes when a Froude number of approximately 0.5 was exceeded. To alleviate this problem on Little Timber Creek, it was proposed to construct two grade-control structures (locally called ditch checks) at locations 547 ft and 1,322 ft above the mouth of the stream, Figure 1. These structures would reduce the bed gradient to an average slope of 8.45 ft/mile. With these structures in place, computations for the design discharge indicated that a maximum average velocity of approximately 7.0 fps would occur, with a corresponding Froude number of 0.43, which was considered compatible with the fat clays (CH) and medium to lean clays (CL) found throughout the Little Timber project reach.

Further studies indicated that a savings of $2,500 could be made over the use of rock ditch checks by using twin steel-sheet pilings at each check, with grouted stone between the pilings, Figures 2 and 3. This plan was adopted. The specifications for the No. 8 gage galvanized steel-sheet piling complied with the provisions of the American Railway Engineering Association Specification 1-4-6. The sheet piling was placed with a pile driver. The stone in the 36-in.-thick horizontal layer across the channel bottom, Figure 3, was specified to have a maximum diameter of 24 in. and to be graded from coarse to fine with a maximum weight of 1,000 lb, and with 40 percent of the stone weighing more than 100 lb. The 18 in. maximum diameter riprap used on the side slopes was specified to weigh approximately 250 lb with no stone weighing more than 300 lb. The stone was placed with a grapple bucket. After placement the riprap surface was grouted. The cost for placement of the two grade-control structures is not separable from the total project cost. The two structures were collectively selected as a Section 32 existing site.

In October 1977, an inspection was made of the condition of the improvements on Little Timber Creek and found that the structures had been performing their intended function; however, a scour hole had developed downstream from ditch check 1, Figure 4, and the grouted rock revetment had been undermined and was breaking off at ditch check 2, Figure 5. The damage to the grouted rock has been repaired by the city of Frankfort. The Section 32 inspection team from the Waterways Experiment Station inspected the Frankfort site on 19 September 1978 and found the grade-control structures to be performing as designed. Inspections through 10 March 1981 indicate that the ditch checks continue to prevent channel degradation (Figure 6).

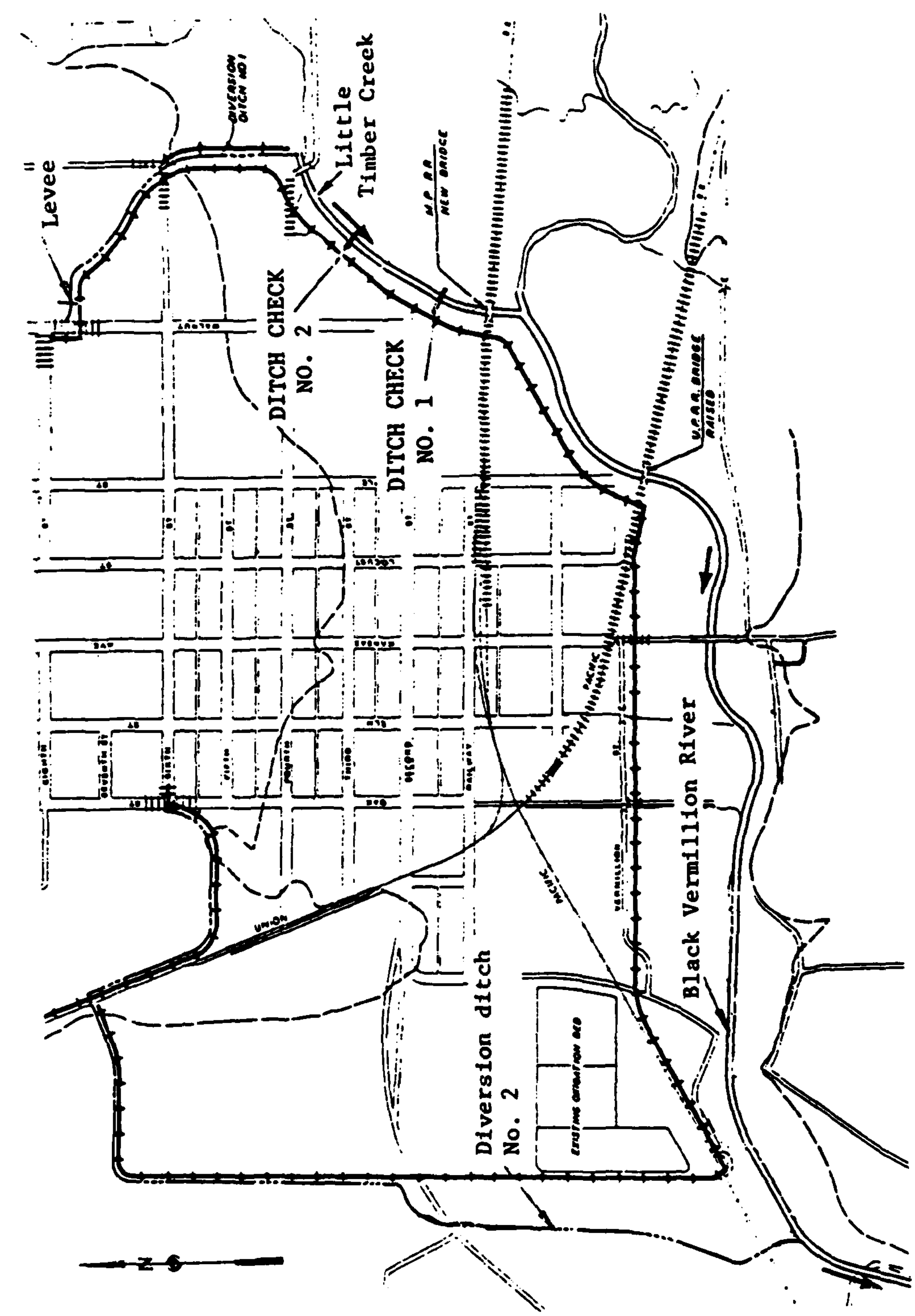

Figure 1. Little Timber Creek at Frankfort, Kansas

ITEM 10-2

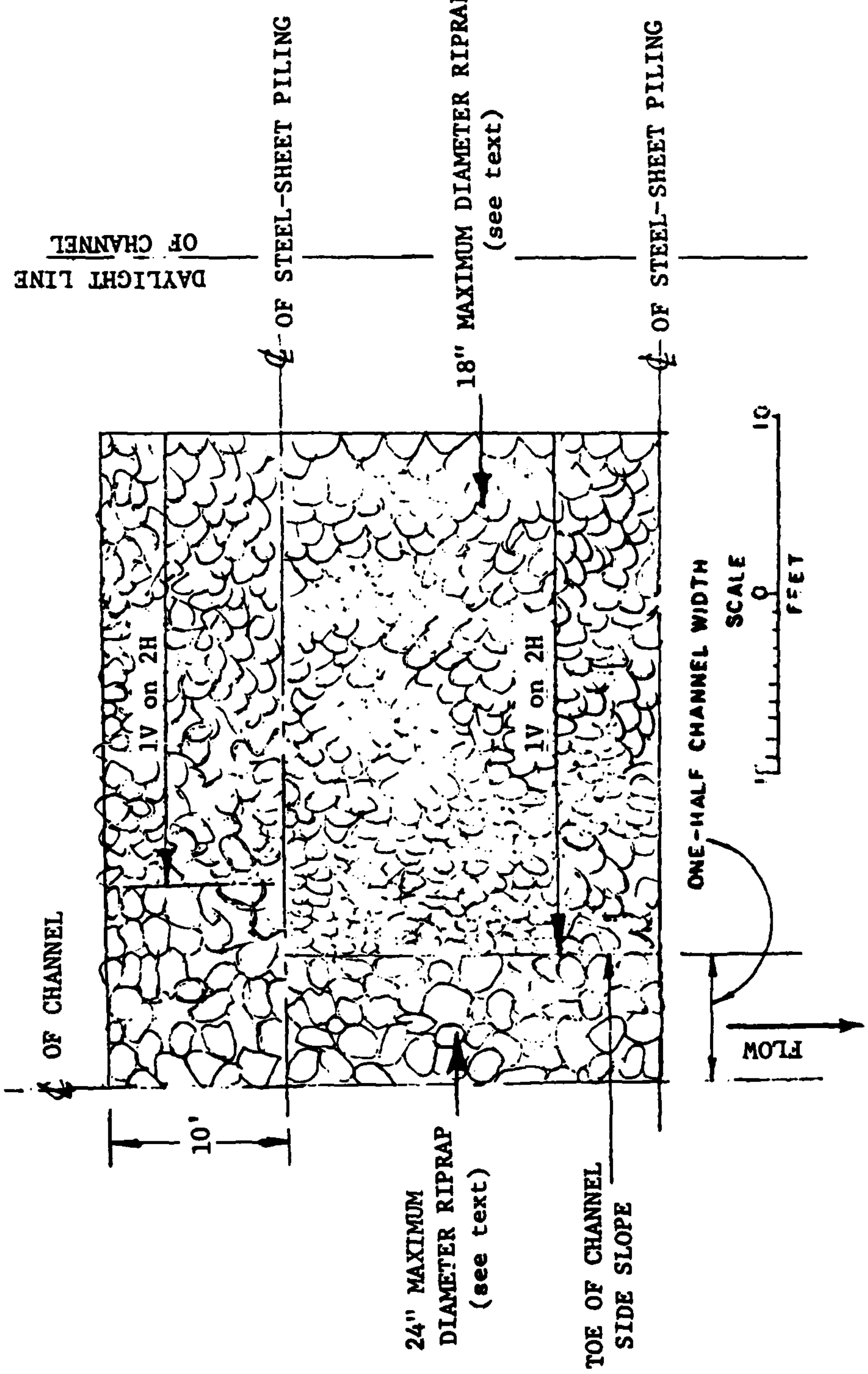

Figure 2. Little Timber Creek at Frankfort, Kansas. Half plan of ditch checks.

ITEM 10-3

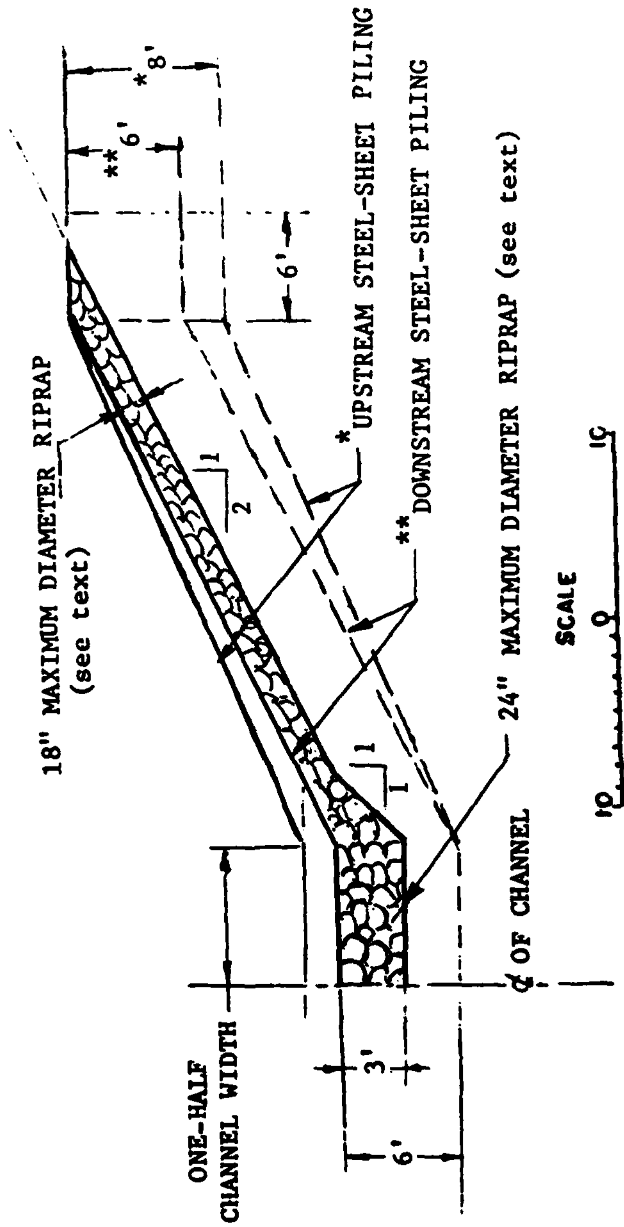

Figure 3. Little Timber Creek at Frankfort, Kansas. Half section of ditch checks looking upstream.

Figure 4. Little Timber Creek at Frankfort, Kansas.
View of scour hole downstream from ditch check 1.

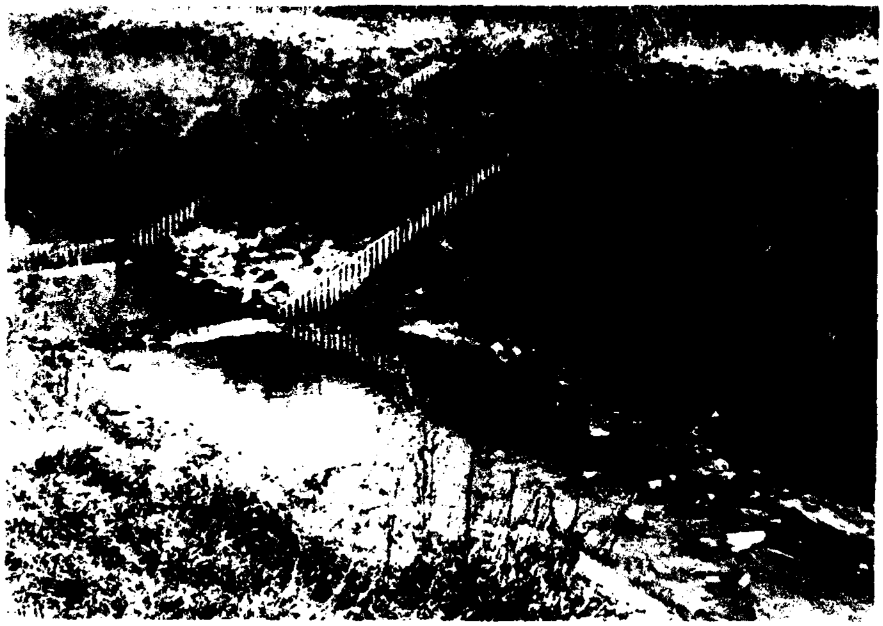

Figure 5. Little Timber Creek at Frankfort, Kansas.
Downstream view showing loss of grouted rock from ditch check 2.

ITEM 10-5

Figure 6. Little Timber Creek at Frankfort, Kansas.
Ditch check condition as of 10 March 1981.

ITEM 10-6

MUD CREEK
LAWRENCE, KANSAS

Streambank Erosion Control Evaluation and Demonstration Act of 1974
Section 32 Program - Work Unit 2

EVALUATION OF EXISTING BANK PROTECTION WORKS

(1) Location

Stream Mud Creek River Mile 0.8-2.0 Side ______

Local Vicinity ______ Lat N39°0' Long W 95°10'

At/Nr City Lawrence County Leavenworth State KS Cong Dist 2

CE Office Symbol MRK Responsible Agency Corps of Engineers

Site Map Sources Kansas City District, Corps of Engineers

Land Use Homes, farming, and light industry

(2) Hydrology at or Near Site

Stage Range NA to ______ ft; *Period of Record 19__ to 19__.*

Discharge Range NA to ______ cfs; Velocity Range ______ to ______ fps

Sediment Range NA to ______ tpd; *Period of Record 19__ to 19__.*

Bank-full Stage ______ ft; Flow 19,250 cfs; Average Recurrence Interval ______ yr

Bank-full Flow Velocity: Average 9 fps; Near Bank ______ fps

Comments Discharge and velocity represent design conditions.

(3) Geology and Soil Properties

Bank (USCS) Lean clay (CL) Bed (USCS) Lean clay (CL)

Data Sources Corps of Engineers test borings

Groundwater Bank Seepage ______

Overbank Drainage ______

Comments ______

(4) Construction of Protection

Need for Protection Channel straightening resulting in steeper gradient and increased velocity.

Erosion Causative Agents See above

Protection Techniques Four-grade control structures.

General Design Sheet piling and rock sills.

Project Length 10,000 ft; Construction Cost $ 165,000 Mo/Yr Completed 1978

(5) Maintenance

Experienced Flows (Stage, cfs, Date) ______

Repairs and Costs (Item, Cost, Data) ______

Comments: No repairs required since construction in 1978.

(6) Performance Observations and Summary

Monitoring Program Visual inspections.

Documentation Sources Kansas City District, Corps of Engineers

Project Effect on Stream Regime Stabilized channel.

Project Effect on Environment Negligible

Successful Aspects Prevention of channel degradation.

Unsuccessful Aspects None to date.

General Evaluation Functioning as planned.

Recommendations ______

(7) Additional Information, Comments, and Summary

Map No. 11. Riprapped banks with preformed scour hole have prevented formation of lateral erosion below structures.

Attached Items:

11-1 Project summary
11-2 Project location
11-3 Plan view
11-4 Cross-sectional view
11-5 Longitudinal cross sections and photo
11-6 Photographs

Mud Creek at Lawrence, Kansas (Mile 0.8 to 2.0)

Mud Creek is a left-bank tributary of the Kansas River joining the main stem at Lawrence, Kansas (Kansas River mile 46.8). Streamflows in the Mud Creek Basin originate 14 miles due north of Lawrence and move in a southerly direction to a point near Midland, Kansas, about 4 miles north of Lawrence. The stream then flows in a southeasterly direction until it converges with the Kansas River. The total stream length is 20.4 miles, with the gradient of the channel being approximately 6.5 ft/mile. The Mud Creek watershed has a long rectangular shape with a total area of 38 square miles. The watershed upstream from Midland has an area of 30 square miles with topography typical of a small midwestern basis, i.e., a relatively narrow floodplain with steep side slopes. Downstream from Midland, the Kansas River has meandered through the area, apparently eroding away any natural hills along the right bank of the Mud Creek Basin, thus leaving hills adjacent only to the left bank.

During the relatively frequent low discharge floods (less than 4,500 c.f.s.) that occurred before channel improvements, the Mud Creek Basin could contain these floodwaters within its boundary; however, during flood events of 4,500 c.f.s. or greater, excess discharges flowed into the Kansas River floodplain. Although there were a few natural levees and a number of low-elevation artificial levees constructed by local residents, these topographic features and structures were easily breached or outflanked by the high discharges. Two of the largest storms were in June 1966 and June 1967. The 1966 flood flow was computed to have peaked at 8,800 c.f.s. on 13 June based on a unit hydrograph and between 8,00 and 10,000 c.f.s. using high-water marks. Flood and hydrographic data were not collected during the 1967 storm, but local residents estimated that stages were equal to or greater than those of 1966.

There have been no stream gaging records maintained within the Mud Creek Basin; thus hydrologic computations had to be made synthetically

based on flood marks and on records of stream gaging stations located near the basin. No suspended-sediment sample collection stations have been operated in the basin. Estimates indicate that average annual sediment yields over the drainage area are 1,000-3,000 tons per square mile. The surface soils along the banks of Mud Creek are predominantly lean clays (CL).

Flood-control improvements for the Lawrence, Kansas, area were authorized by the 1954 Flood Control Act (Public Law 83-780, as a part of the Missouri River Basin comprehensive plan for flood control. The plan of protection for the Mud Creek Unit consisted of the following major elements:

a. Placement of levees (23,020 ft).

b. Channel improvement (including grade control) from the mouth to the upstream limits of levee construction (6.1 miles).

c. Bridge improvements, including removal of existing bridges and construction of new bridges as required to eliminate potential constrictions.

d. Drainage structures for removal of interior drainage.

e. Stone riprap protection.

At the request of officials of the city of Lawrence, the Mud Creek portion of the Lawrence area project was restudied and as a result the plans were revised. The major changes from the original design were:

a. The three-fourths standard project flood design discharge at the mouth of Mud Creek was increased from 14,000 to 19,250 c.f.s., with a design velocity of 9 fps.

b. Levee and channel improvements were extended upstream an additional 3 miles.

c. Bridge construction was revised to accommodate the increased design discharge.

d. Levee elevations were raised to provide a minimum of 3-ft freeboard to protect against the new design discharge.

The Mud Creek channel improvement and levee work was awarded on 13 May 1976 and completed in July 1978. Upstream from the improvements, conditions have remained essentially the same as the preproject downstream conditions with the channel heavily choked with timber and underbrush.

An essential feature of the Mud Creek Channel improvement project was the construction of four sheet piling and rock sills at miles 0.80, 1.31, 1.55, and 1.96 to provide grade control and prevent channel degradation, Figure 1. These four structures were collectively selected as a Section 32 existing site. Plan and cross-sectional views of the sills are provided in Figures 2, 3, and 4, respectively. The completed structures were placed at an approximate cost of $165,000 (1978).

The steel-sheet piling was required to conform to Military Specification MIL-P-11858, Type II, Section Number Z-27. The sheet piles were interlocking, Figure 5, throughout their entire lengths to the drive depth indicated. The riprap portion of the sheet-piling and rock sill structures were placed on a 6-in.-thick layer of bedding material and 12-in.-thick layer of spalls, Figure 3 and 4. Type C riprap (42 in.) was used in the center of the channel and Type A riprap (18 in.) on the side slopes. The riprap, spalls, and bedding materials were all placed with a dragline.

Stone for the riprap was required to be sound, durable limestone free from cracks, seams, shale partings, and overburden spoil. The riprap was specified to be approximately rectangular in cross section and relatively free from flat and elongated pieces. The quantity of stone having an elongation ratio greater than 3 could not exceed 5 percent by weight. The riprap was graded subject to the following limits.

Weight per Stone lb	Percent of Total Weight Lighter Than
Type A (18 in.)	
300	100
200	80-95
80	30-50
10	0-10
Type C (42 in.)	
4,000	100
3,000	80-95
1,000	30-50
200	0-20

Material for the spalls layer was required to be of tough, durable particles. The total of objectionable material, friable particles, and other foreign matter could not exceed 5 percent by weight. The gradation specified for the spalls was:

Sieve Size	Percent by Weight Passing
10 in.	100
7 in.	75-95
5 in.	40-60
3 in.	20-40
2 in.	0-20

The gradation required for the bedding material was:

Sieve Size	Percent by Weight Passing
6 in.	100
4 in.	75-95
1 in.	40-60
3/8 in.	15-35
No. 4	0-15

At the time of the 1978 inspection by the Waterways Experiment Station team, the project was performing as designed. Additional inspections through March 1981 (Figures 6 and 7) indicate that the structures are intact and no headcutting is evident through the project reach. Riprapped banks of the preformed scour holes have prevented the formation of lateral erosion.

ITEM 11-1
(Sheet 4 of 4)

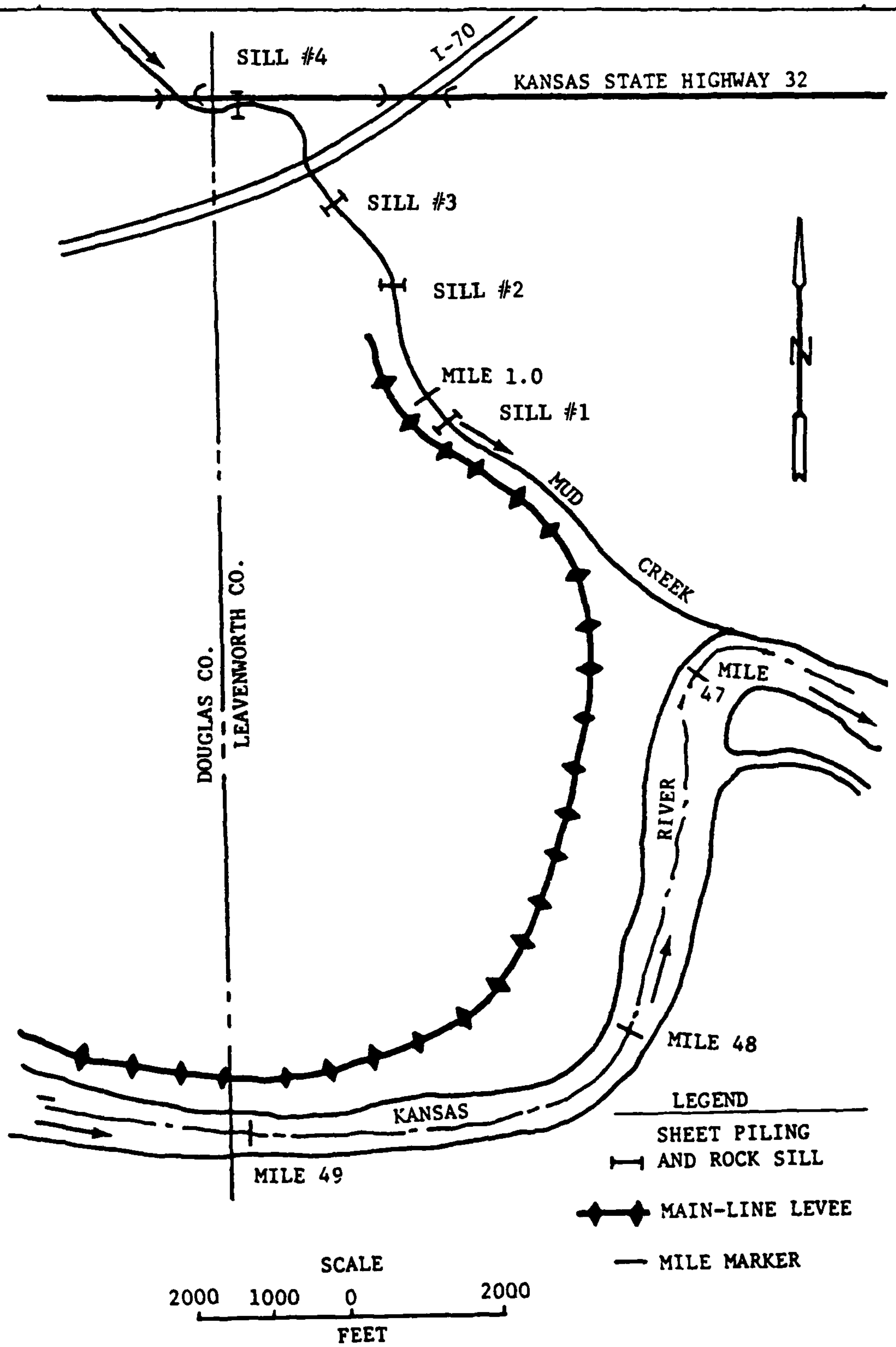

Figure 1 . Mud Creek at Lawrence, Kans.

ITEM 11-2

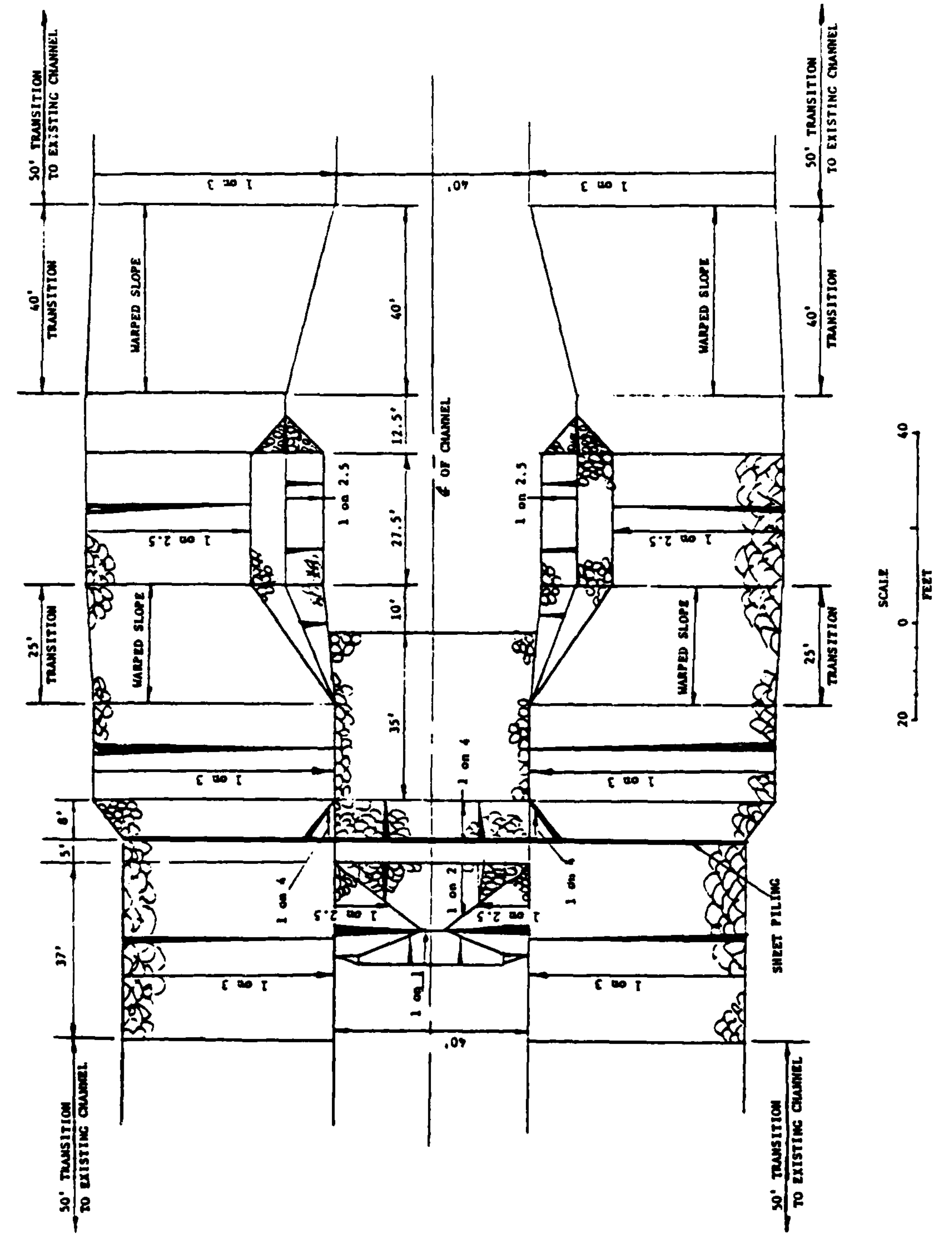

Figure 2. Mud Creek at Lawrence, Kans. Plan view at sheet-piling and rock sill structure.

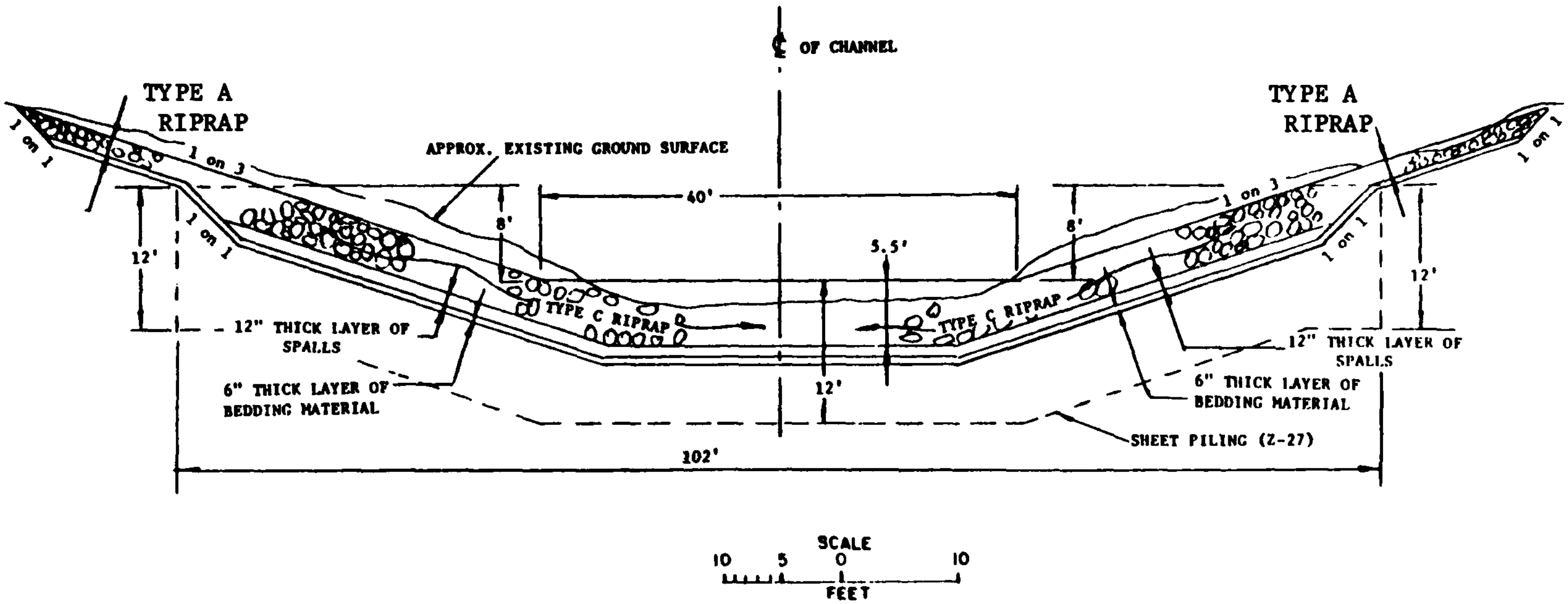

Figure 3. Mud Creek at Lawrence, Kans. Cross-sectional view of sheet-piling and rock sill structure.

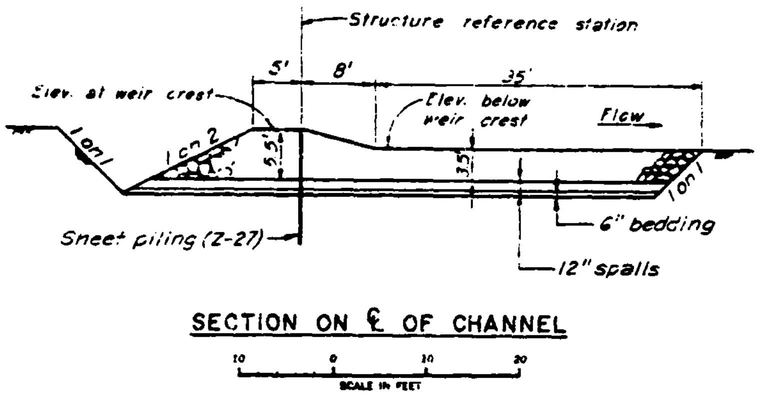

Figure 4. Mud Creek at Lawrence, Kans. Longitudinal cross section of sheet-piling and rock sill.

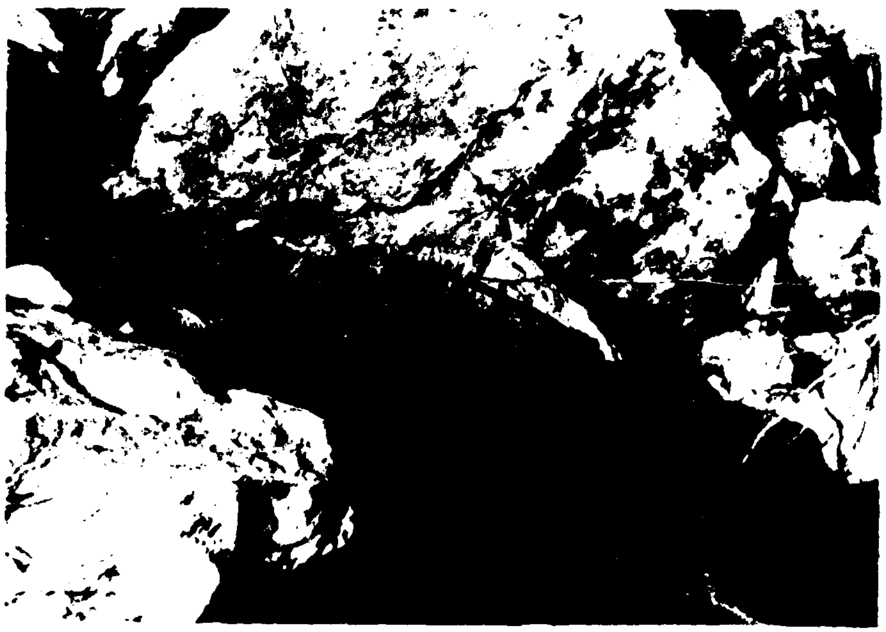

Figure 5. Mud Creek at Lawrence, Kans. Interlocking sheet piles.

Figure 6. Mud Creek at Lawrence, Kans. Sheet piling and rock sill at mile 1.55 (11 March 1981)

Figure 7. Mud Creek at Lawrence, Kans. Sheet piling and rock sill at mile 0.80 (11 March 1981)

LITTLE BLUE RIVER
INDEPENDENCE, MISSOURI

Streambank Erosion Control Evaluation and Demonstration Act of 1974
Section 32 Program - Work Unit 2

EVALUATION OF EXISTING BANK PROTECTION WORKS

(1) Location

Stream Little Blue River River Mile 7.5 Side

Local Vicinity Lat N39°06' Long W94°18'

At/Nr City Independence County Jackson State MO Cong Dist 4

CE Office Symbol MRK Responsible Agency Corps of Engineers

Site Map Sources Corps of Engineers, Kansas City District

Land Use Homes, farming, and industrial

(2) Hydrology at or Near Site

Stage Range 0 to 23.2 ft; Period of Record 1948 to 1981.

Discharge Range 0 to 17,000 cfs; Velocity Range 0 to 9 fps

Sediment Range 0 to 253,144 tpd; Period of Record 1971 to 1981.

Bank-full Stage 23.5 ft; Flow 18,000 1/ cfs; Average Recurrence Interval 100 yr

Bank-full Flow Velocity: Average 9 fps; Near Bank fps

Comments Bed gradient 3 ft/mile. 1/ Improved channel.

(3) Geology and Soil Properties

Bank (USCS) Sandy silts to lean clays Bed (USCS) Sandy, silty clay

Data Sources Corps of Engineers test borings.

Groundwater Bank Seepage NA

Overbank Drainage

Comments

(4) Construction of Protection

Need for Protection To maintain bed-gradient equilibrium and prevent slope failures due to channel degradation.

Erosion Causative Agents Channel shortening with resultant increase in bed gradient.

Protection Techniques Sheet piling and rock sills in low flow channel.

General Design One row of interlocking sheet pile normal to channel with stone up and downstream to prevent erosion.

Project Length ft; Construction Cost $ Mo/Yr Completed 12/78

(5) Maintenance

Experienced Flows (Stage, cfs, Date) 17,000 cfs, September 1977

Repairs and Costs (Item, Cost, Data) None to date.

Comments:

(6) Performance Observations and Summary

Monitoring Program Visual inspection, resurveyed in January 1981.

Documentation Sources

Project Effect on Stream Regime

Project Effect on Environment

Successful Aspects

Unsuccessful Aspects

General Evaluation Sills appear to be working as planned. Low flow channel remains stable.

Recommendations

(7) Additional Information, Comments, and Summary

Map No. 12.

Attached Items:

12-1 Project summary and location
12-2 Typical plan view
12-3 Photograph and cross section
12-4 Comparison project profiles

Little Blue River at Independence, Missouri

The Little Blue River is a right bank tributary of the Missouri River, joining the main stem at mile 339.5 (1960 adjustment) 20 miles downstream from Kansas City, Missouri. The Little Blue Basin is 33 miles long, with a maximum width of 13 miles. The total drainage area of the basin is 224 square miles of which 90 percent is in Jackson County, Missouri, with the remainder being in Cass County, Missouri. The lower 7.4 miles of the Little Blue River are confined to an improved channel between the right bank bluffs of the Missouri River floodplain and the Unit R-351 tieback levee of the Missouri River Levee System.

The Little Blue River Basin is frequently subject to flooding from high-intensity rainstorms, mostly during the months of April through October. Flood stage at the Lake City gage has been exceeded 21 of the 23 years since records have been kept. The gradual encroachment of the Kansas City metropolitan complex into the basin has significantly raised the flood damage potential. To mitigate this threat, a project for channel improvement and reservoir construction was authorized for the Little Blue Basin by the 1968 Flood Control Act, Public Law 90-483 (part of the comprehensive plan for the Missouri River Basin). This legislation provides for channel improvement (in four stages) from mile 7.4 through mile 22.4, and the construction of Longview Dam upstream from the main-stem channel improvements and Blue Springs Lake Dam on the East Fork of the Little Blue River. The channel improvement feature incorporates two types of channels: a low-flow channel that follows much of the bed of the existing Little Blue River, and a high-flow channel 5 ft in elevation above the existing waterway to handle flood discharges. When the project is completed, the natural channel length will be shortened from 22 miles to 15 miles for the high-flow channel and to 18 miles for the low-flow channel.

The Stage I channel improvement contract was awarded on 20 December 1975 (Figure 1) and was completed on 5 December 1978. Three Section 32 existing sites were selected in the Stage I reach:

ITEM 12-1
(Sheet 1 of 4)

a. Stone riprap on the side slopes of the low-flow channel.

b. Sheet piling and rock sills.

c. Compacted clay on the berm and side slopes of the high-flow channel.

The structures placed at each of these sites were designed to withstand a 100-year flood. This report is concerned with the sheet piling and rock sills in the low-flow channel.

Since March 1948, the U.S. Geological Survey (USGS) has maintained a stream gaging station at the Missouri State Highway 78 Bridge (reported as the Lake City gage). The daily discharges of record are: maximum 17,000 c.f.s. (during the September 1977 flood); mean 133 c.f.s.; and minimum no flow on several occasions. The channel through Stage I is designed for 18,000 c.f.s. (100-year flood). The maximum observed stream velocity is 9 ft/sec which occurred during the September 1977 flood. A suspended-sediment sample station has been operated at this site by MRK since October 1971; for the period of record the maximum load is 253,144 tons/day, the mean 426 tons/day, and the minimum 0 tons/day. The maximum annual load of record is 374,933 tons (water year 1977); the average annual load is 155,556 tons. Average sediment load in this reach consists of 6 percent sand, 51 percent silt, and 43 percent clay. The average annual sediment yield upstream from the Stage I reach is 750 to 1,000 tons per square mile. Soil types vary from clays in the upper end of the Stage I reach (CL and CH) to sands at the lower end (SM, SP, and sandy ML).

Shortening of the natural channel through the Stage I reach required placement of five sheet-piling and rock-sill structures* to maintain bed-gradient equilibrium (3 ft/mile). The sheet piling conformed to Military Specification MIL-P-11858, Type II, Section Z-27. The interlocking piling

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* A sheet-piling and rock sill structure was in place at sta 737+00 prior to the Stage I channel improvements.

was driven with a pile hammer and then trimmed, Figures 2 and 3. The rock used for structure consisted of Type D riprap placed on a 12-in.-thick layer of spalls material, which in turn rested on a 6-in.-thick layer of bedding material (Figure 4).

The Type D riprap was required to meet the following gradation standards:

Weight per Stone lb	Percent of Total Weight Lighter than
2,000	100
1,500	85-95
500	30-50
70	0-15

The 12-in.-thick layer of spalls material was specified to meet the following requirements:

Sieve Size	Percent by Weight Passing
12 in.	100
8 in.	75-95
4 in.	40-60
1/2 in.	5-25

As with the Type D riprap, the spalls were required to be approximately rectangular in cross section, to be relatively free from thin slabby pieces, and to have an elongation ratio greater than 3. The 6-in.-thick layer of bedding material placed under the spalls was required to meet the following gradation:

Sieve Size	Percent by Weight Passing
4 in.	100
3 in.	75-95
3/4 in.	40-60
3/8 in.	20-40
No. 4	5-25

Material not passing the 3/4-in. sieve was specified to be reasonably free from flat elongated particles and deleterious substances.

The in-place cost of the sheet piling and rock sills was not separable from the total project cost.

ITEM 12-1
(Sheet 3 of 4)

The flood of record in September 1977 (17,000 c.f.s.) caused no damage to any of the structures in the Stage I reach. Longitudinal surveys were made in January 1981 of the sheet piling and rock sills at stations 764+00 and 773+00. The 1981 channel bottom is superimposed on the as-built drawings in Figures 5 and 6. The surveys indicate that the channel is reaching equilibrium.

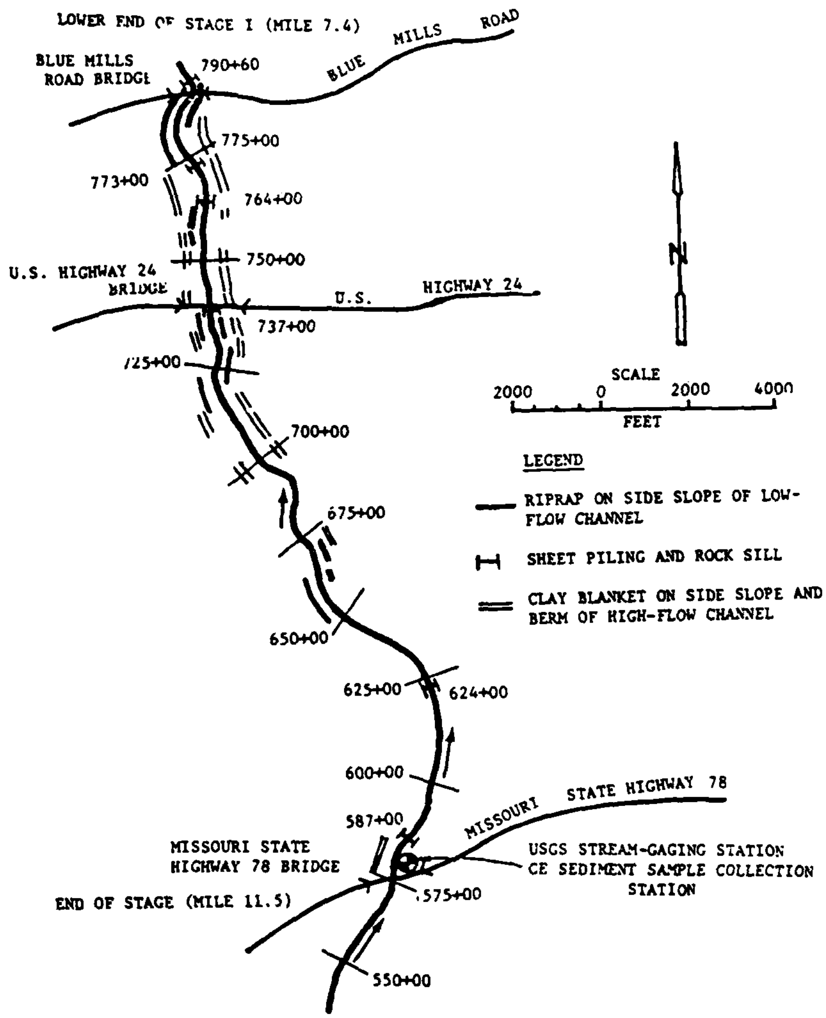


Figure 1. Little Blue River channel improvement at Independence, Mo., Stage I (sta 577+00 to 788+96)

ITEM 12-1
(Sheet 4 of 4)

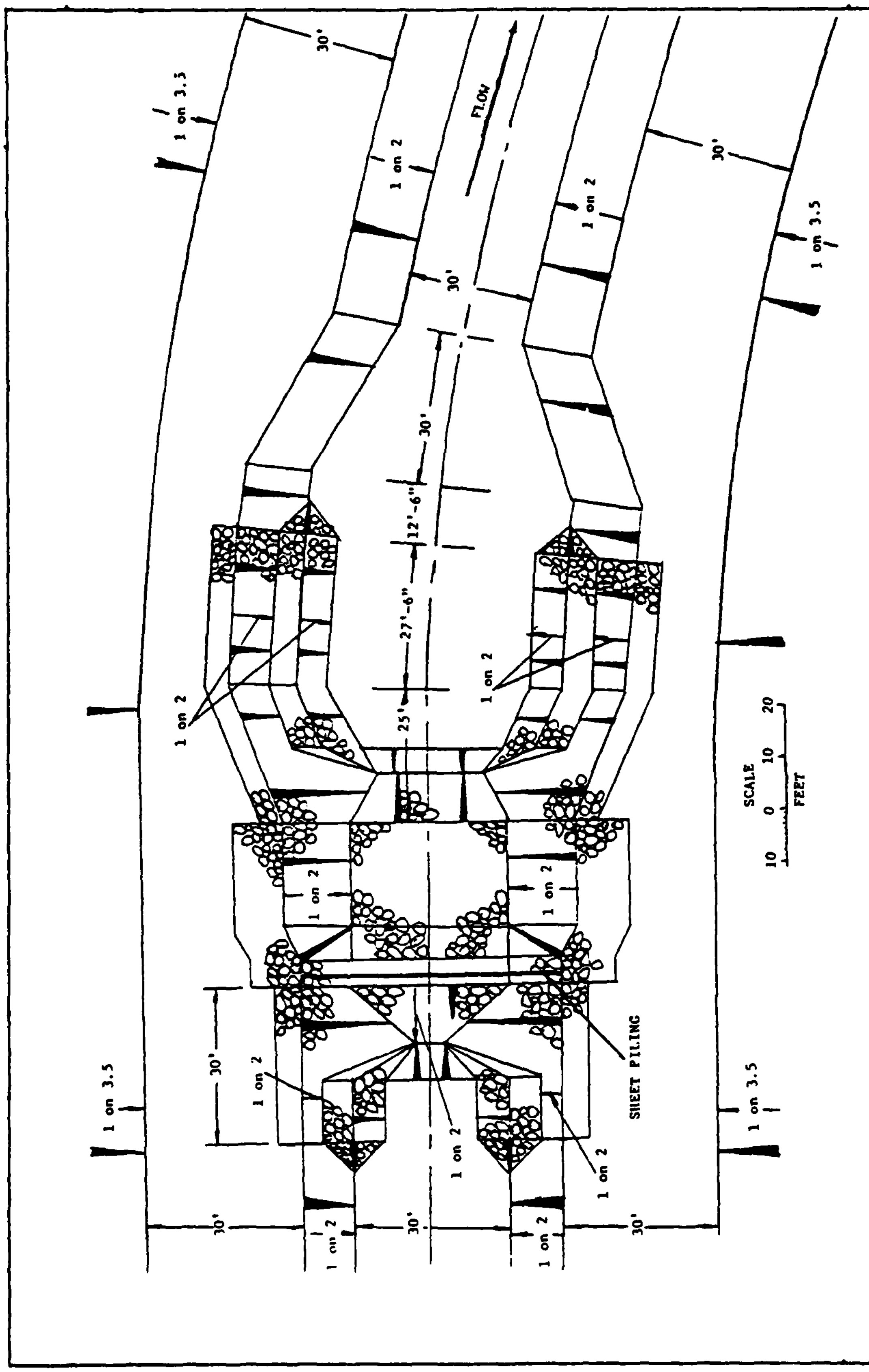


Figure 2. Little Blue River at Independence, Mo. Typical plan view of sheet piling and rock sills

ITEM 12-2

Figure 3. Little Blue River at Independence, Mo. Interlocking sheet piling used for construction of sills.

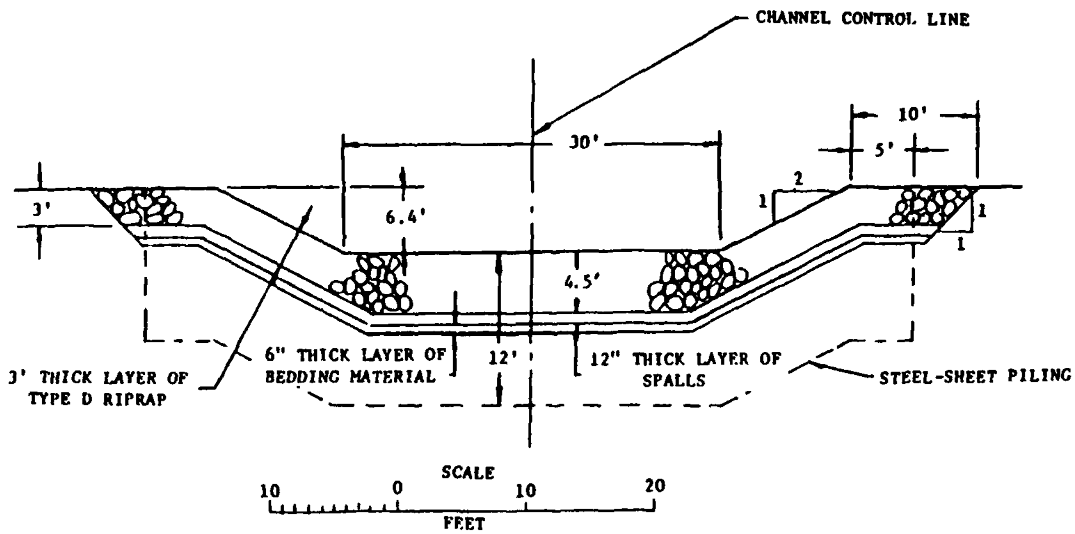
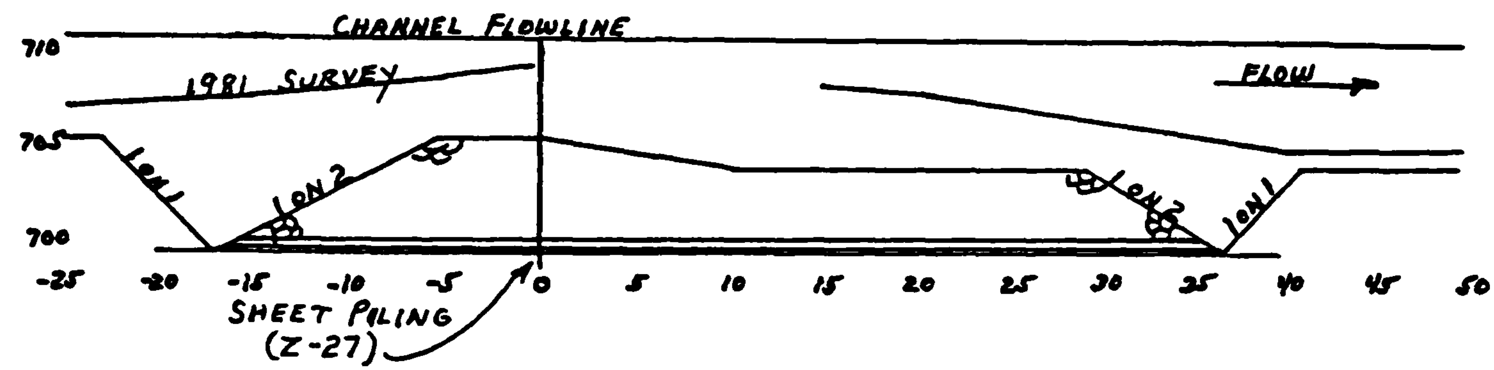


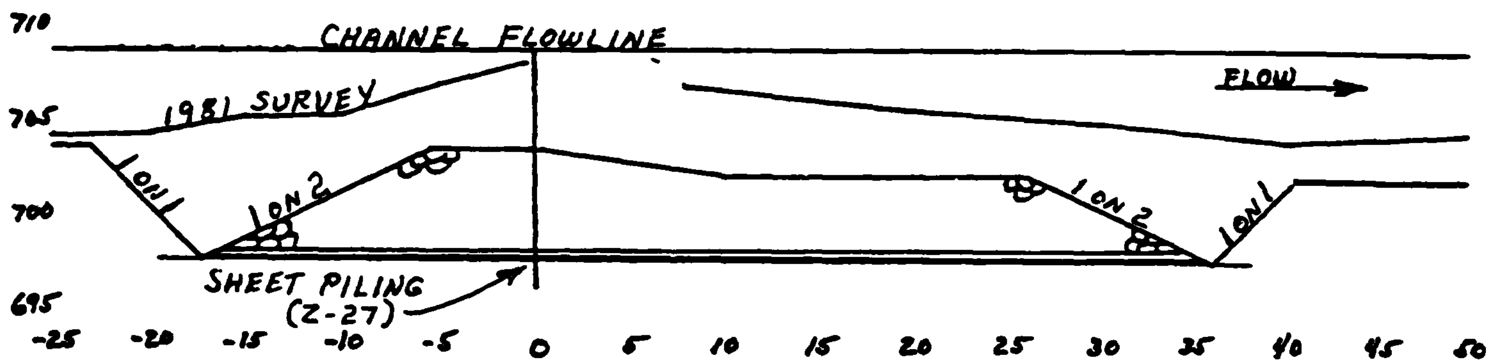
Figure 4. Little Blue River at Independence, Mo. Cross-sectional view of sheet piling and rock sills.

ITEM 12-3



SECTION ON CHANNEL CONTROL LINE

Figure 5. Sheet pile and rock sill at station 764+00.



SECTION ON CHANNEL CONTROL LINE

Figure 6. Sheet pile and rock sill at station 773+00.

Little Blue River at Independence, Mo. Comparison of channel bottom as-built and as determined by survey of January 1981.

LITTLE BLUE RIVER
INDEPENDENCE, MISSOURI

Streambank Erosion Control Evaluation and Demonstration Act of 1974
Section 32 Program - Work Unit 2

EVALUATION OF EXISTING BANK PROTECTION WORKS

(1) Location

Stream Little Blue River River Mile 7.4-11.5 Side Both

Local Vicinity ______ Lat N39°06' Long W94°18'

At/Nr City Independence County Jackson State MO Cong Dist 4

CE Office Symbol MRK Responsible Agency Corps of Engineers

Site Map Sources Corps of Engineers, Kansas City District

Land Use Homes, farming, and industrial

(2) Hydrology at or Near Site

Stage Range 0 to 23.3 ft; Period of Record 1948 to 1981.

Discharge Range 0 to 17,000 cfs; Velocity Range 0 to 9 fps

Sediment Range 0 to 253,144 tpd; Period of Record 1971 to 1981.

Bank-full Stage 23.5 1/ft; Flow 18000 1/cfs; Average Recurrence Interval 100 yr

Bank-full Flow Velocity: Average 9 fps; Near Bank ______ fps

Comments Bed gradient 3 ft/mile. 1/Improved channel.

(3) Geology and Soil Properties

Bank (USCS) Sandy silts to lean clays. Bed (USCS) Sandy, silty clay.

Data Sources Corps of Engineers test borings.

Groundwater Bank Seepage NA

Overbank Drainage Localized uncontrolled rain runoff.

Comments ______

(4) Construction of Protection

Need for Protection Excavation of high flow channel exposed noncohesive materials in lower 3 miles of project.

Erosion Causative Agents High flows over noncohesive materials.

Protection Techniques Seeded clay blanket

General Design 12 in. clay blanket placed, then seeded and mulched.

Project Length ______ ft; Construction Cost $ * Mo/Yr Completed 12/78

*Construction costs cannot be isolated from overall project costs.

(5) Maintenance

Experienced Flows (Stage, cfs, Date) 17,000 cfs. September 1977

Repairs and Costs (Item, Cost, Data) None to date.

Comments:

(6) Performance Observations and Summary

Monitoring Program Visual inspection.

Documentation Sources Corps of Engineers, Kansas City District

Project Effect on Stream Regime

Project Effect on Environment

Successful Aspects Has prevented general erosion of high flow channel berm and banks.

Unsuccessful Aspects In areas of uncontrolled overbank drainage, some scour occurred.

General Evaluation Satisfactory performance overall.

Recommendations Overbank drainage should be collected and routed to paved drains.

(7) Additional Information, Comments, and Summary

Map No. 13.

Attached Items:

13-1 Project summary and location

13-2 Typical section and photograph

13-3 Project photographs

Little Blue River at Independence, Mo. (Mile 7.4 to 11.5)

The Little Blue River is a right bank tributary of the Missouri River, joining the main stem at mile 339.5 (1960 adjustment) 20 miles downstream from Kansas City, Mo. The Little Blue Basin is 33 miles long, with a maximum width of 13 miles. The total drainage area of the basin is 224 square miles of which 90 percent is in Jackson County, Mo., with the remainder being in Cass County, Mo. The lower 7.4 miles of the Little Blue River are confined to an improved channel between the right bank bluffs of the Missouri River floodplain and the Unit R-351 tieback levee of Missouri River Levee System.

The Little Blue River Basin is frequently subject to flooding from high-density rainstorms, mostly during the months of April through October. Flood stage at the Lake City gage has been exceeded 21 of the 23 years since records have been kept. The gradual encroachment of the Kansas City metropolitan complex into the basin has significantly raised the flood damage potential. To mitigate this threat, a project for channel improvement and reservoir construction was authorized for the Little Blue Basin by the 1968 Flood Control Act, Public Law 90-483 (part of the comprehensive plan for the Missouri River Basin). This legislation provides for channel improvement (in four stages) from mile 7.4 through mile 22.4, and the construction of Longview Dam upstream from the mainstem channel improvements and Blue Springs Lake Dam on the East Fork of the Little Blue River. The channel improvement feature incorporates two types of channels: a low-flow channel that follows much of the bed of the existing Little Blue River, and a high-flow channel 5 ft in elevation above the existing waterway to handle flood discharges. When the project is completed, the natural channel length will be shortened from 22 miles to 15 miles for the high-flow channel and to 18 miles for the low-flow channel.

ITEM 13-1
(Sheet 1 of 4)

The Stage I channel improvement contract was awarded on 20 December 1975, Figure 1, and was completed on 5 December 1978. Three Section 32 existing sites were selected in the Stage I reach. These were:

a. Stone riprap on the side slopes of the low-flow channel.

b. Sheet piling and rock sills.

c. Compacted clay on the berm and side slopes of the high-flow channel.

The structures placed at each of these sites were designed to withstand a 100-year flood. This report is concerned with the compacted clay blanket.

Since March 1948, the U.S. Geological Survey (USGS) has maintained a stream gaging station at the Missouri State Highway 78 Bridge (reported as the Lake City gage). The daily discharges of record are: maximum 17,000 c.f.s. (during the September 1977 flood); mean 133 c.f.s.; and minimum no flow on several occasions. The improved channel through Stage I is designed for 18,000 c.f.s. (100-year flood). The maximum observed stream velocity was 9 ft/sec which occurred during the September 1977 flood. A suspended-sediment sample station has been operated at this site since October 1971. The maximum load for the period of record is 253,144 tons/day, the mean 426 tons/day, and the minimum 0 tons/day. The maximum annual load of record is 374,933 tons (water year 1977); the average annual load is 155,556 tons. Average sediment load in this reach consists of 6 percent sand, 51 percent silt, and 43 percent clay. The average annual sediment yield upstream from the Stage I reach is 750 to 1,000 tons per square mile. Soil types vary from clays in the upper end of the Stage I reach (CL and CH) to sands at the lower end (SM, SP, and sandy ML).

Pre-project boring logs indicated that much of the proposed high-flow channel side-slope and berm surfaces in the lower 3 miles of the

project would be in noncohesive material. To minimize bank erosion in this reach, the sand was replaced by an impervious clay blanket. The blanket material was specified to be of low permeability, consisting of clays (CH) and (CL), and to be free of plant growth, roots, and humus. The composition of the impervious material was such that a minimum of 50 percent of the soil particles by weight must pass a U.S. Standard No. 200 sieve. The minimum liquid limit of the material was specified to be 40. After bank preparation, the material was placed in one lift to a final minimum thickness of 12 in., Figure 2. Compaction was accomplished by two passes of a crawler tractor. To assure channel stability, the final design for Stage I required that 11,547 lin. ft of the high-flow channel side slopes and berms be covered by a clay blanket between Sta 671+00 and 784+00, representing 51 percent of the total length of the banks between these stations, Figure 1. In addition, a 700-ft segment of the right bank below the Missouri State 78 Highway Bridge was blanketed. The impervious blanket placement required 25,553 cu yd of material at an in-place cost of $2.00/cu yd (1975). The total area covered was 16.5 acres.

The clay blankets were seeded with a grass mixture as follows:

Seed Type	lb/acre
Tall Fescue	15
Domestic Rye	6
Smooth Brome	15
Reed Canary	10
Total	46

After the seeding was completed, mulch was placed over the seeded areas. The bid cost (1974) for materials (seed and mulch) and the planting operation was $825/acre.

The flood of record in September 1977 (17,000 c.f.s.) caused no significant damage to any of the structures in the Stage I reach.

ITEM 13-1
(Sheet 3 of 4)

However, as a result of uncontrolled overbank drainage, clay blanket erosion occurred at some locations, Figure 3. These areas have been repaired by the construction of two gabion drop structures, Figure 4, and a number of grouted gutters, Figure 5. Several areas of erosion of the clay blanket on the high flow berm will be repaired by a series of low rock dikes across the berm.

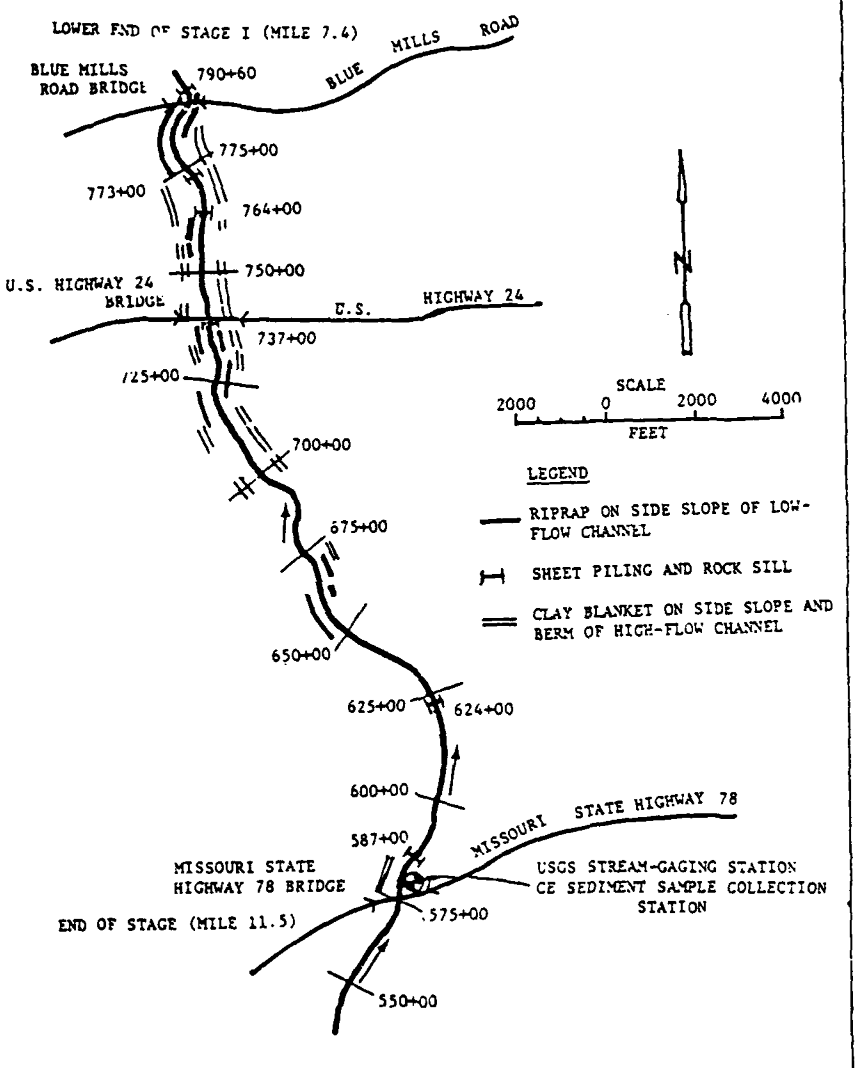


Figure 1. Little Blue River channel improvement at Independence, Mo., Stage I (sta 577+00 to 788+96)

Item 13-1
(Sheet 4 of 4)

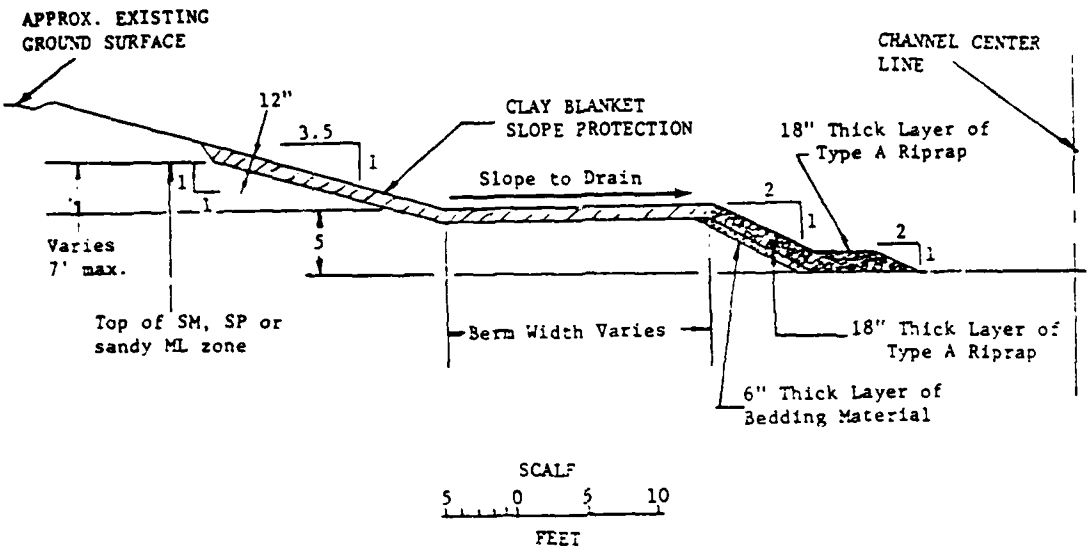


Figure 2. Little Blue River at Independence, Mo. Typical section of clay blanket side-slope and berm protection.

Figure 3. Little Blue River at Independence, Mo. As a result of uncontrolled overbank drainage, clay blanket erosion has occurred at some locations (May 1977)

ITEM 13-2

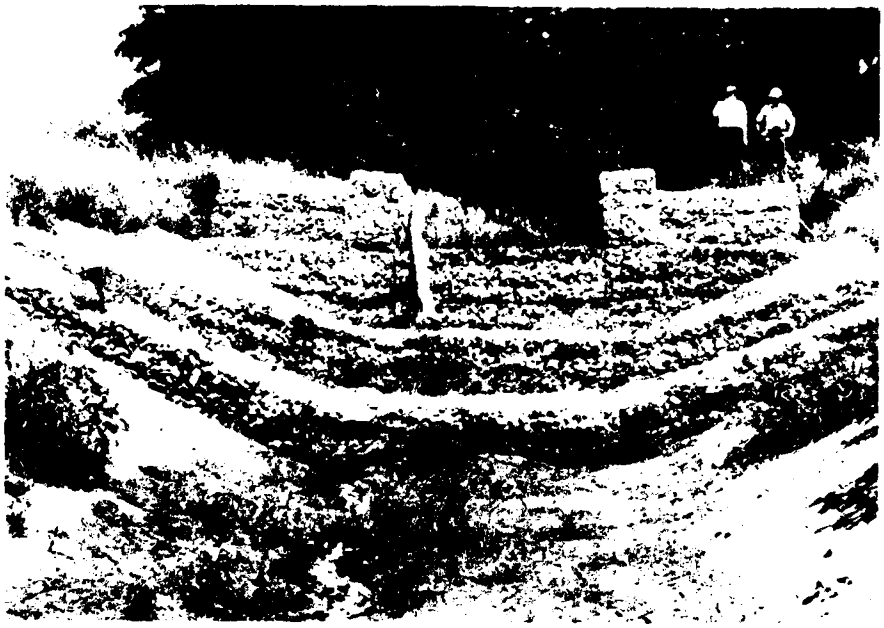


Figure 4. Little Blue River at Independence, Mo. Gabion drop structure to control overbank drainage.

Figure 5. Little Blue River at Independence, Mo. Grouted gutter to control overbank drainage.

ITEM 13-3

BIG BLUE RIVER
NEAR MARYSVILLE, KANSAS

Streambank Erosion Control Evaluation and Demonstration Act of 1974
Section 32 Program - Work Unit 2

EVALUATION OF EXISTING BANK PROTECTION WORKS

(1) Location

Stream Big Blue River River Mile 76.4 Side Right

Local Vicinity Central Kansas Lat N39°45' Long W96°42'

At/Nr City Marysville County Marshall State KS Cong Dist 2

CE Office Symbol MRK Responsible Agency Corps of Engineers

Site Map Sources Corps of Engineers, Kansas City District

Land Use Homes and farming

(2) Hydrology at or Near Site

Stage Range ______ to ______ ft; Period of Record 19 32 to 19 81.

Discharge Range 1 to 57,000 cfs; Velocity Range NA to NA fps

Sediment Range 1 to 357,800 tpd; Period of Record 19 59 to 19 72.

Bank-full Stage ______ ft; Flow 15,000 cfs; Average Recurrence Interval ______ yr

Bank-full Flow Velocity: Average NA fps; Near Bank NA fps

Comments Bed gradient 0.5 ft/mile

(3) Geology and Soil Properties

Bank (USCS) Fine sands, silt, Gravel lenses Bed (USCS) Sand, gravel, silt.

Data Sources Visual classification

Groundwater Bank Seepage ______

Overbank Drainage Field drain between dikes 2 and 3.

Comments ______

(4) Construction of Protection

Need for Protection To prevent damage to county road and right abutment of county bridge.

Erosion Causative Agents High flows and channel alignment.

Protection Techniques Wire fence on RR rail posts with rock dike tiebacks.

General Design RR rail posts set 4'8" in ground on 8' centers and cable connected, wire mesh tied on, rock dike tiebacks to high bank.

Project Length 900 ft; Construction Cost $ 39,895 Mo/Yr Completed 7/77

(5) Maintenance

Experienced Flows (Stage, cfs, Date) 26.3, 26,000 cfs, 3/3/79; 25.5, 25,200 cfs, 7/22/78; 25.1, 24,500 cfs, 5/7/78; 22.8, 20,500 cfs, 3/23/79

Repairs and Costs (Item, Cost, Data) See (7)

Comments:

(6) Performance Observations and Summary

Monitoring Program Visual inspections, resurveyed in 1980

Documentation Sources Corps of Engineers, Kansas City District

Project Effect on Stream Regime Smoothed the alignment around a bend.

Project Effect on Environment Sediment deposited behind fence and between dikes has revegetated and improved terrestrial habitat.

Successful Aspects Smoother alignment and improved wildlife habitat.

Unsuccessful Aspects Fence undermined through 70 percent of reach and fell forward and is lying on riverbed.

General Evaluation Overall the project is functioning as planned with exception of fence revetment.

Recommendations A small windrow of rock at the base of fence would prevent toe failure.

(7) Additional Information, Comments, and Summary

Map No. 14. In early 1979 high flows destroyed the bridge and county does not intend to replace it. There is no need to maintain the stabilization structure.

Attached Items:

14-1 Project summary
14-2 Project plan view
14-3 Photographs of completed project
14-4 Photographs of damage
14-5 Photographs 4 yrs after construction

Big Blue River Near Marysville, Kansas (Mile 76.4)

Prior to 1973, flooding on the Big Blue River had caused considerable erosion of the right bank upstream from the county bridge located one-half mile west of the town of Schroyer, Kansas (5 miles southwest of Marysville, Kansas). At that time, the bridge was used extensively as a farm-to-market road and U.S. mail route. An inspection of the bridge and its approaches by the County Engineer after flooding in October 1973 indicated that flanking of the structure was imminent. As a result the Marshall County Commission requested assistance from the Corps of Engineers under Section 14 of the Flood Control Act of 1946. In response, the following actions were considered: (a) no Federal action; (b) arrest the bank erosion at the present structure; and (c) construct another bridge in the immediate area. The analysis concluded that with the no Federal action alternative, bank erosion would eventually result in failure of the bridge. Failure of the structure would create a hardship in this rural area, since this is the only bridge crossing in an 11-mile reach. Action (b) was considered the more feasible of the construction alternatives since the cost of building a new bridge was estimated at $215,000 (1975).

There are no discharge or suspended-sediment data for the vicinity of the Schroyer Bridge. However, data are available for the USGS gaging station at Barneston, Nebr. (mile 107.2). The daily discharges of record (1932 to the present) are maximum 57,000 c.f.s., mean 771 c.f.s., and minimum 1 c.f.s. MRK operated a suspended-sediment sample collection station at the same location from September 1959 through September 1972. Daily suspended-sediment loads of record were: maximum 357,800 tons (2 March 1966), mean 3,943 tons, and minimum 1.0 ton (16 August 1964). The maximum annual suspended-sediment load of record was 3,619,067 tons (water year 1965); the average annual suspended-sediment load was 1,439,107 tons. The average annual sediment yield in the area upstream from the Schroyer Bridge is 500 to 1,000 tons/square mile. Soils in this reach consist mostly of fine sands and silts in the bed and banks, with some gravel in

the bed and in lenses on the bank. The streambed gradient through this reach is 0.5 ft/mile.

Due to funding delays and right-of-way problems, construction of the Schroyer Bridge protection project was not undertaken until June 1977; the project was completed the following month. The final design consisted of 700 ft of fencing attached to railroad rail posts with three rock-dike tiebacks and 200-ft of rock revetment at the upstream end of the fencing, Figure 1. The purpose of this configuration was not only to redirect the flow, but also to induce deposition of sediment and thus reestablish the bank line along the fence.

The upstream revetment and the three dikes were constructed by placement of 1,500 tons of quarry-run stone. The construction specifications required that no more than 5 percent of the stone could be under 1/2-in. diameter, with the maximum stone weight being 500 lb. The upstream revetment was placed at the angle of repose of the stone. The zero crest width dikes were 6 ft in height with no specified bottom width; dike numbers 1-3 were 45, 150, and 125 ft in length, respectively, Figure 1.

The 10-ft-long fence posts were fabricated from salvage railroad rail and set 4 ft 8 in. in the ground on an 8-ft spacing. The posts were stabilized with three 1/2-in. galvanized-steel cables, which traversed the fence line at the bottom, middle, and top of the posts, respectively. The cables were passed through holes burned through the rails, and then permanently positioned with a cable clamp on either side of the hole. The fencing was then attached to the rails and cables with two strands of twisted No. 12 galvanized-steel wire. The base of the fence was buried to a depth of 1 ft, leaving 5 ft above the finished earth surface. The final cost (1977) of the project was $39,895.21. The completed bank protection works were designated as a Section 32 existing site.

After completion, the project experienced three periods of high flows in 1977. Some minor damage occurred when a large tree was deposited on the fence during one of these discharges. The maximum estimated flow during these three events was 16,000 c.f.s. High flows in July 1978 undermined a section of the fence, causing it to fail, Figure 3. By March 1981, the fence was severely damaged between dikes 1 and 2 and below dike 3. Between dikes 2 and 3 the fence is completely gone. The use of a wire fence revetment at this location was an experiment to determine the effect of waterborne debris on a fence revetment. Some minor damage to the fence fabric occurred, but most of the damage was due to undercutting of the fence. A small windrow of rock at the base of the fence would probably have prevented this failure.

With the exception of the fence failure noted above, the project is performing as designed, Figures 4, 5, and 6. Deposition is occurring behind the dikes and vegetation is becoming reestablished. The bridge failed during high flows and an ice run in early 1979; the steel span structure was washed downstream to the next point bar. Marshall County has no plans to replace the structure unless Federal funding becomes available. Therefore, no repair has been performed on the fence.

ITEM 14-1
(Sheet 3 of 3)

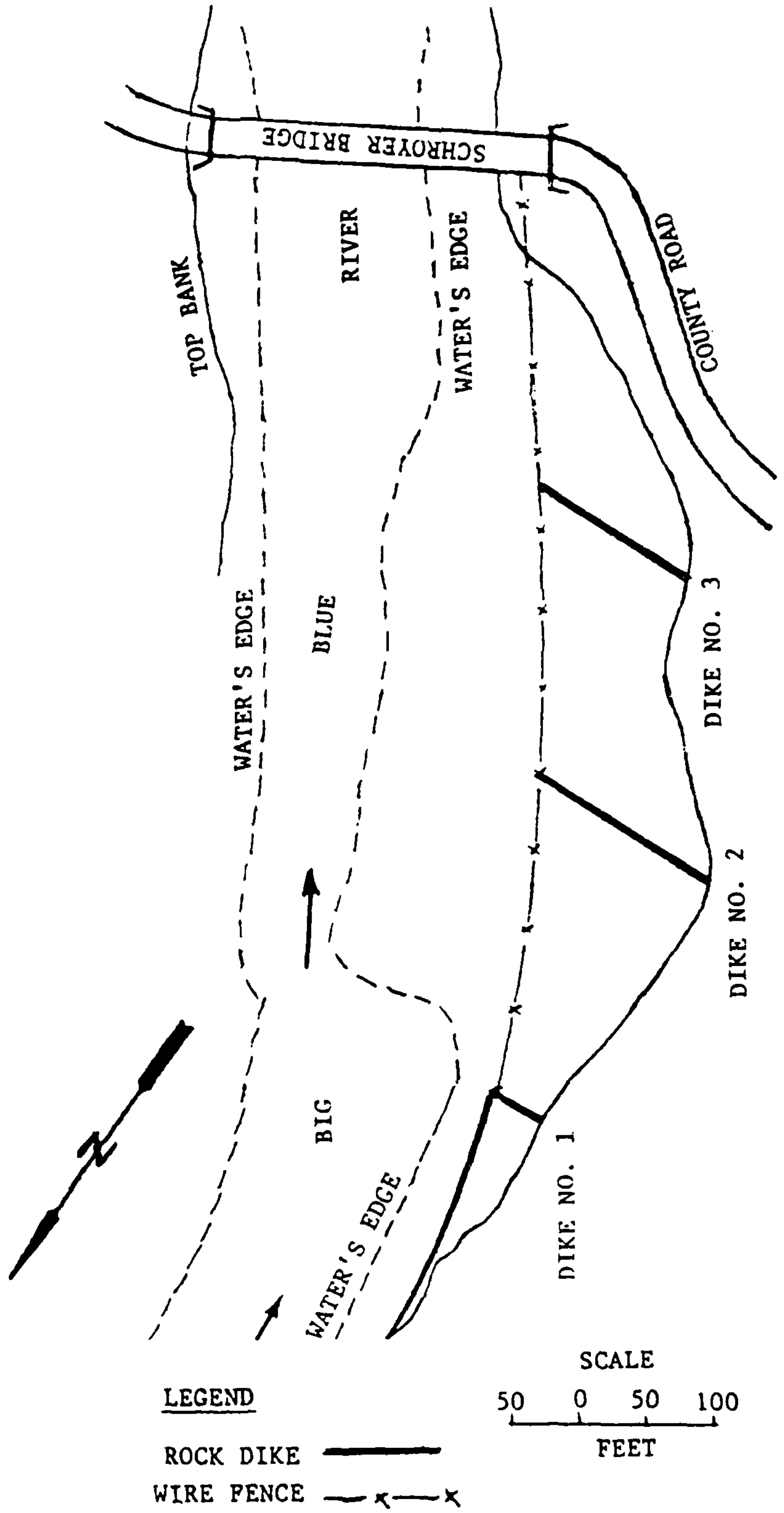


Figure 1. **Big Blue River near Marysville, Kans. Plan view of rock dikes and wire-fence revetment.**

ITEM 14-2

a. Downstream view of wire fence revetment. Schroyer Bridge is in the background.

b. Wire fence revetment viewed toward end of dike 3.

Figure 2. Big Blue River near Marysville, Kans. Two views of the completed project. (July 1977)

ITEM 14-3

Figure 3. Big Blue River near Marysville, Kans. High flows in July 1978 undermined a section of the fence causing it to fail. This photograph was taken from the Schroyer Bridge (15 August 1978)

Figure 4. Big Blue River near Marysville, Kans. Upstream view from Schroyer Bridge following high flows in August 1977.

ITEM 14-4

Figure 5. Big Blue River near Marysville, Kans. Condition as of 10 March 1981. View is upstream from former location of the Schroyer Bridge.

Figure 6. Big Blue River near Marysville, Kans. Condition as of 10 March 1981. View is downstream from upper end of stone fill revetment. Note fill and vegetation.

ITEM 14-5

102 RIVER
BEDFORD, IOWA

Streambank Erosion Control Evaluation and Demonstration Act of 1974
Section 32 Program - Work Unit 2

EVALUATION OF EXISTING BANK PROTECTION WORKS

(1) Location

Stream 102 River River Mile ______ Side Right & left

Local Vicinity ______ Lat N40°40' Long W 94°43'

At/Nr City Bedford County Taylor State Iowa Cong Dist 5

CE Office Symbol MRK Responsible Agency Corps of Engineers

Site Map Sources Corps of Engineers, Kansas City District

Land Use Water and sewage plants and urban development

(2) Hydrology at or Near Site

Stage Range ______ to ______ ft; Period of Record 1959 to 1981.

Discharge Range 0 to 9,980 cfs; Velocity Range 8 to 13 fps 2/

Sediment Range NA to NA tpd; Period of Record 19__ to 19__.

1/ Bank-full Stage ______ ft; Flow 9,500 cfs; Average Recurrence Interval 15-20 yr

2/ Bank-full Flow Velocity: Average 12 fps; Near Bank ______ fps

Comments ______

(3) Geology and Soil Properties

Bank (USCS) Lean and sandy clays, fat clay lenses Bed (USCS) Same as banks

Data Sources Corps of Engineers

Groundwater Bank Seepage ______

Overbank Drainage ______

Comments ______

(4) Construction of Protection

Need for Protection Channel improvement project completed in Oct '67 was severely damaged by flooding in Oct '73

Erosion Causative Agents High flows and channel degradation

Protection Techniques Fabriform mat

General Design Bank shaped with quarry-run stone then covered with double-walled woven nylon fabric filled with fine-aggregate concrete

Project Length 3,000 ft; Construction Cost $ 126,000 Mo/Yr Completed 7/77

(5) Maintenance

Experienced Flows (Stage, cfs, Date) 4,930 cfs, 10 June 1974; 7,600 cfs 1977; 3,000 cfs, March 1979; 3,970 & 8,280 cfs July 1979; 6,460 & 3,970 cfs June 1980.

Repairs and Costs (Item, Cost, Data) Emergency repairs by hired labor in Mar 79 at cost of $32,900. Further repairs by contract in Feb 80 at cost of $38,200. Repairs were by gabions and grouted rock.

Comments: 1979 emergency repairs necessitated by grade control and additional inspections.

(6) Performance Observations and Summary

Monitoring Program Semiannual visual inspections and additional inspections following periods of high flows.

Documentation Sources Corps of Engineers, Kansas City District

Project Effect on Stream Regime

Project Effect on Environment

Successful Aspects Provides good upper slope protection.

Unsuccessful Aspects When toe fails or channel degrades, the Fabriform will fail due to inflexibility.

General Evaluation Can provide adequate bank protection if toe is not undermined.

Recommendations Should only be used when toe is stabilized.

(7) Additional Information, Comments, and Summary

Map No. 15. The original grade-control structures failed, allowing the channel to degrade which led to piping of fill from beneath the Fabriform which started breaking off due to lack of support.

Attached items:
15-1 Project summary and location
15-2 General cross section
15-3-5 Plan views
15-6-11 Project photographs

102 River (East Fork) at Bedford, Iowa

The East Fork of the 102 River rises in the hills of northeastern Taylor County, Iowa, flows past the county seat at Bedford, and then joins the West and Middle Forks near Hopkins, Mo., to form the 102 River. The East Fork was reworked by local landowners both upstream and downstream of Bedford in 1946 and 1947. Approximately 2 miles of channel was improved northeast of Bedford beginning at a point 1,000 ft above the Iowa State Highway No. 2 Bridge (Figure 1) and extending upstream; the channel was also improved downstream of Bedford to Hopkins, Mo. A short, 300-ft-long segment of improvement was also made in the city of Bedford. Including the work done by the Corps of Engineers, the length of channel between Bedford and Hopkins has been reduced from about 21 miles to 12 miles.

Discharge data have been taken at the USGS gaging station near Bedford since September 1959 (2.4 miles downstream from the State Street Bridge (Figure 1). Daily discharges of record through the present are: maximum 9,980 c.f.s. (October 1973), mean 49.9 c.f.s., and minimum no flow. No reported suspended-sediment samples have been taken on the East Fork of the 102 River. Average annual sediment yields in the vicinity of Bedford are 3,000 to 6,000 tons/acre. Lean and sandy clays (CL) with some fat clay (CH) are found in both the bed and banks of the 102 River at Bedford.

Corps of Engineers assistance in controlling flood flows on the East Fork at Bedford, Iowa, was initiated in 1966 under authority of Section 205 of the 1943 Flood Control Act, as amended. The first contract was awarded on 16 August 1966; the project was completed on 16 October 1967. The improvement consisted of straightening and widening the channel through Bedford and downstream to a point 2 miles south of the city. The project provided for 1V-on-3H side slopes, a 45-ft bottom width, and a bed gradient modification from 0.84 to 7.13 ft/mile which increased the

design capacity of the improved channel to 9,500 c.f.s. All trees and brush along the channel side slopes and 50 ft landward of the top of bank were removed. Riprap was placed on both banks upstream and downstream of the low head dam (Figure 1); in addition, stone revetment was placed on the right bank from Sta 62+00 to 67+50. The completed project provided protection for 100 urban acres and 300 acres of agricultural lands. The project was completed and transferred to the city of Bedford for operation and maintenance on 29 November 1967.

High flows (over 5,000 c.f.s.) occurred in April 1969. At Sta 62+00 through 67+50 a band of riprap and bedding about 20 ft wide was swept off the bank through the middle third of the slope. General degradation was found to be occurring along the entire length of the channel. A plan to rebuild the eroded slope, replace the riprap and bedding material, and stabilize the channel bottom with a drop structure at Sta 61+60 was developed. Authority to proceed with the construction was received on 23 October 1972. On 29 December 1972, high flows damaged sheeting that had been driven at the drop structure construction site; in addition, the channel degraded 6 ft through this reach. As a result, the structure had to be redesigned, and work was not completed until 12 September 1973.

High flows in October 1973 accelerated erosion adjacent to three existing facilities. Although channel degradation and bank erosion had occurred prior to the flood, the increased degradation and erosion worsened to such an extent at these locations that safety of the facilities was classified as critical. The Kansas City District specified the following emergency measures (Figure 1):

a. Slope repair and stabilization of the right bank adjacent to the Bedford Water Treatment Plant.

b. Slope repair and stabilization of the left bank at the State Street Bridge.

c. Slope repair and stabilization of the left bank at the Bedford Sewage Treatment Plant.

The Kansas City District chose to stabilize the channel side slopes at these three problem areas using Fabriform, manufactured by Construction Techniques, Inc., Cleveland, Ohio, as shown in Figures 2 through 5. Fabriform is a double-walled woven nylon material which is filled with fluid fine-aggregate concrete. The fabric was specified to be of the filter point type (8 in. between points), thus providing capability for drainage. Prior to placement of the Fabriform mattress, the banks were shaped with quarry-run stone (no gradation specified) as shown in Figures 2 and 6. After placement of the mattress on the prepared bank, grout was pumped into the Fabriform. The grout was required to have the following specifications per cubic yard: Portland cement 900 to 1,000 lb, aggregate 2,200 to 2,000 lb, and water 570 to 610 lb. Air entrainment was specified to be in the range 3 to 6 percent of the total mixture.

The contract for the revetment placement was let on 25 February 1974 and work was concluded on 22 May of the same year. The completed protection works, Figures 7 - 9, required 31,000 sq ft of Fabriform and 5,300 tons of quarry-run stone. The blankets were keyed at the top of the bank with an earth-fill anchor trench, Figure 2. These three revetments, whose total cost (1974) was $124,000, were collectively designated as a Section 32 existing site.

A storm event in excess of 8 in. of rainfall occurred on 9-10 June 1974; a mean daily discharge of 4,930 c.f.s. was recorded at the gaging station during this event. A 27 June 1974 inspection indicated that the downstream 25-ft portion of the fabric-covered area at the water treatment plant had received considerable damage resulting from erosion of slabby rock from under the toe supporting the grout-filled fabric on the channel bottom. This erosion extended to a depth of approximately 5 ft and 10 to 20 ft riverward of the intersection of the slope with the channel bottom. The major damage was at the downstream end of the revetment where a fabric seam was torn 5 ft upslope. The State Street and sewage treatment plant revetments were intact and no damage was noted. A further inspection on

4 November 1976 indicated that further erosion under the revetment at the water treatment plant was in progress.

The Waterways Experiment Station (WES) inspection team visited this site on 21 September 1978. Although high flows had undercut the Fabriform at several locations, resulting in pieces breaking off at the toe, Figure 10, or subsidence of sections, Figure 11, most of the revetments were generally in good condition, Figure 12. During the March 1979 flood (2,000 c.f.s. maximum daily flow), the drop structure, Figures 1 and 5, was undercut and failed, Figure 13. In an attempt to protect the upstream reach, a temporary dam was constructed at the site of the structure; the construction proceeded from the right bank across to the left bank. This caused a concentration of flow on the left bank and against the Fabriform mattress before closure was made. As a result, nearly all of the fines were washed out of the rock fill under the Fabriform in this vicinity, creating a "pipe" under the Fabriform that was parallel to the streamflow. This resulted in a relatively large erosion hole on the left bank under the Fabriform. As a result, the undercut Fabriform had to be broken out and rock fill placed in the void.

An inspection of the mattress at the water treatment plant during the same period indicated that a large cavity had formed under the Fabriform up to top of bank, Figure 14; this was a potentially serious situation because the corner of the plant was only 10 to 15 ft from top of bank. An inspection conducted on 31 July 1979 indicated that a considerable portion of the Fabriform adjacent to the plant had been displaced. The city of Bedford had placed several gabions on the exposed bank which probably prevented a total undercutting of the toe. The revetment was repaired in February 1980 with grouted rock and gabions. Under the same contract a temporary grouted rock and gabion structure was placed adjacent to the sewage treatment plant, upstream from the original drop structure. The repaired revetment at the water treatment plant successfully withstood high flows in June 1980; however, the drop structure adjacent to the

sewage treatment plant was lost. As a result, some of the Fabriform was undermined and collapsed into the cavity. High flows in June 1980 also damaged the Fabriform revetment at the State Street Bridge. This damage consisted of undermining and removal of a section of Fabriform adjacent to the bridge pier. See Figures 15 and 16.

A contract was awarded in March 1981 for construction of a concrete baffled chute type drop structure downstream of the sewage treatment plant to replace the structure that was lost. This should control the head-cutting that has occurred.

1/ Original project design conditions. Degradation has increased channel capacity to more than 15,000 cfs.

2/ Existing degraded conditions.

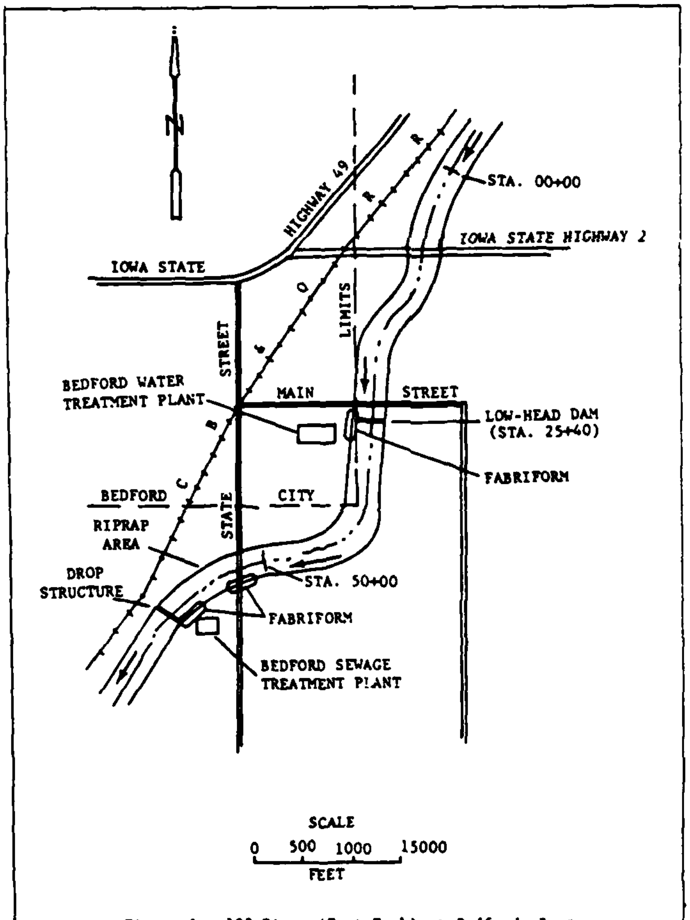


Figure 1. 102 River (East Fork) at Bedford, Iowa.

ITEM 15-1
(Sheet 5 of 5)

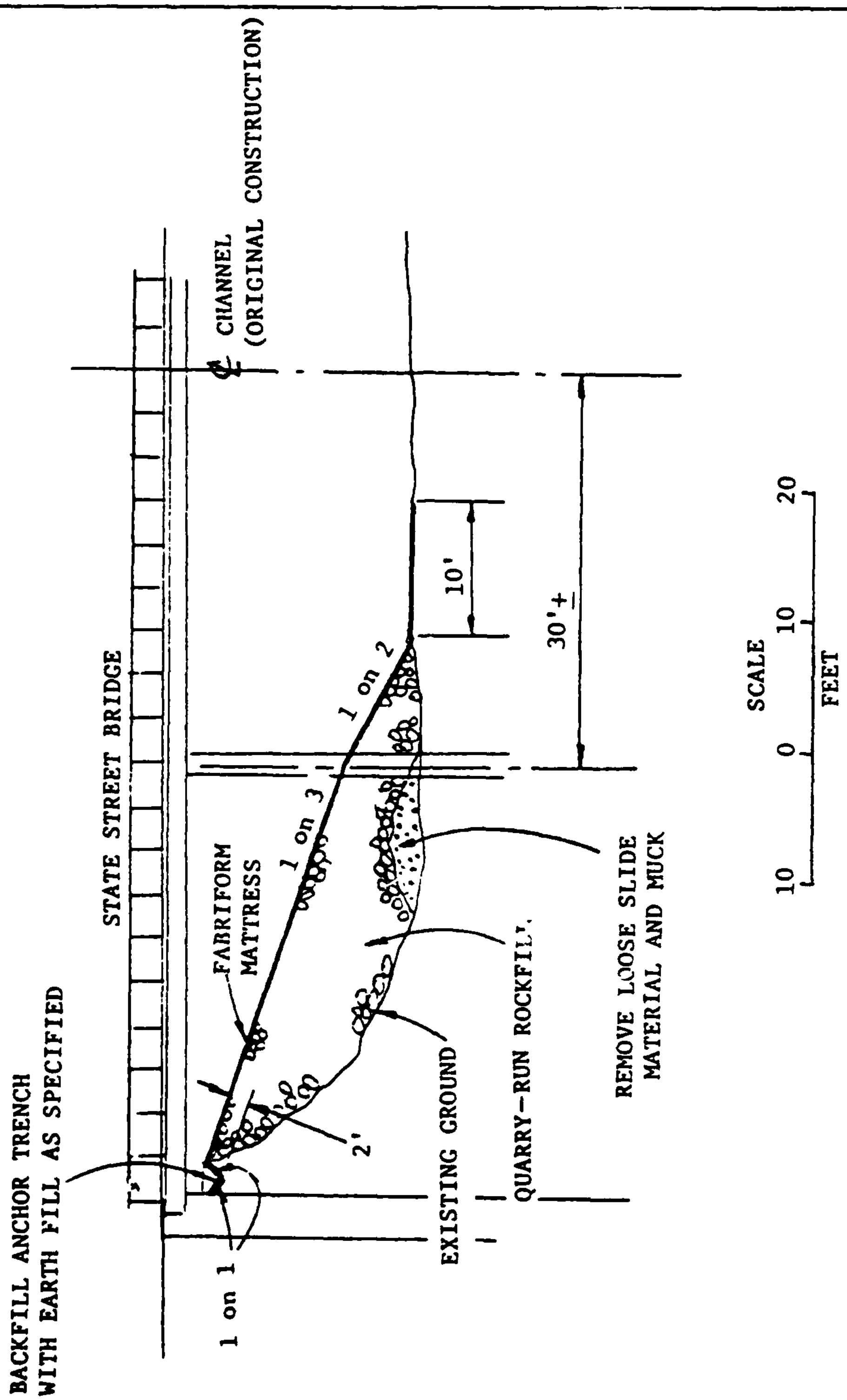


Figure 2. 102 River (East Fork) at Bedford, Iowa. Fabriform revetment on left bank at State Street Bridge (sta 52+42)

ITEM 15-2

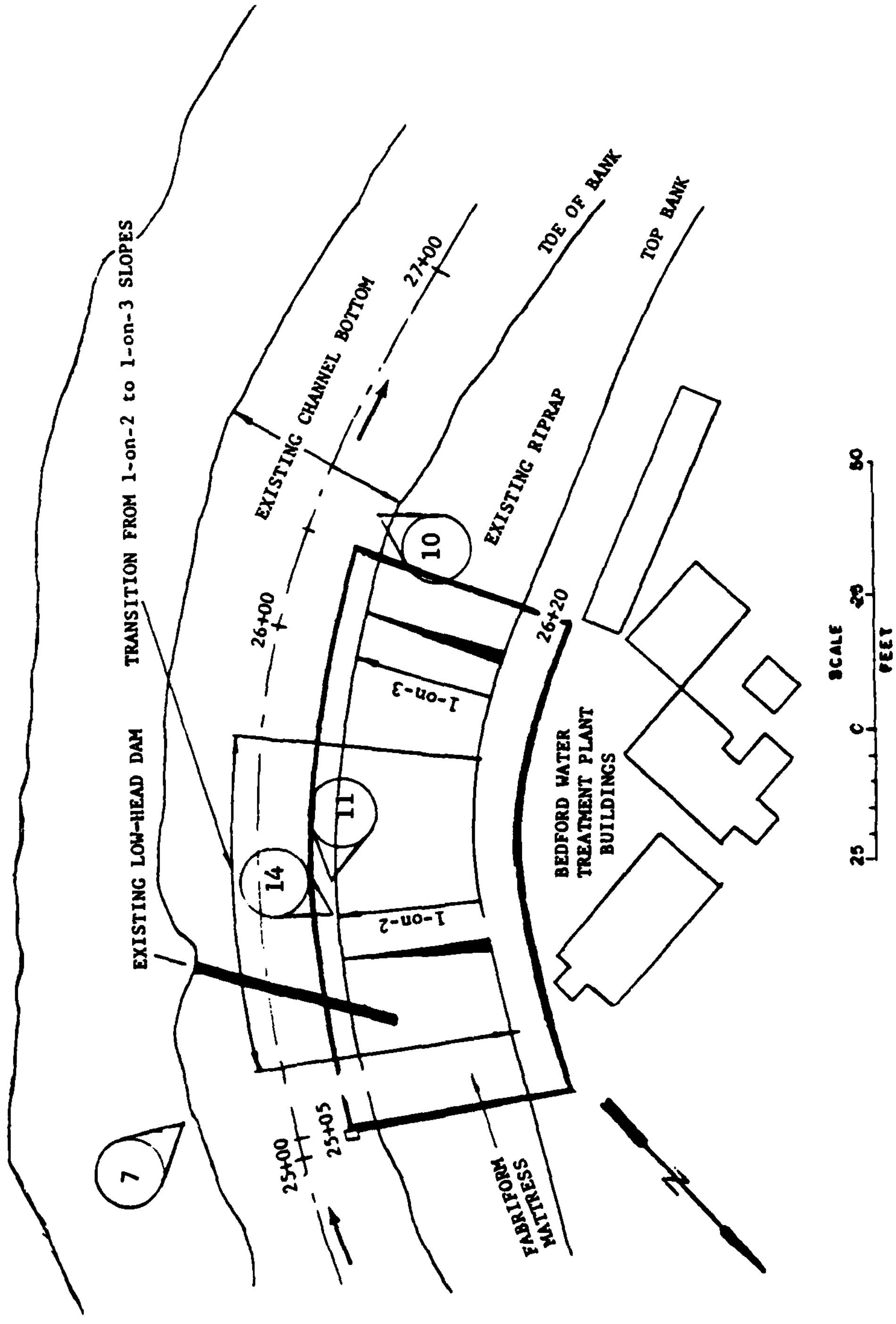


Figure 3. 102 River (East Fork) at Bedford, Iowa. Plan view of Fabriform revetment at Bedford Water Treatment Plant (sta 25+75). Orientation of field-of-view and location from which photograph was taken are represented by ◯▷ for Figures 7, 10, 11 and 14.

ITEM 15-3

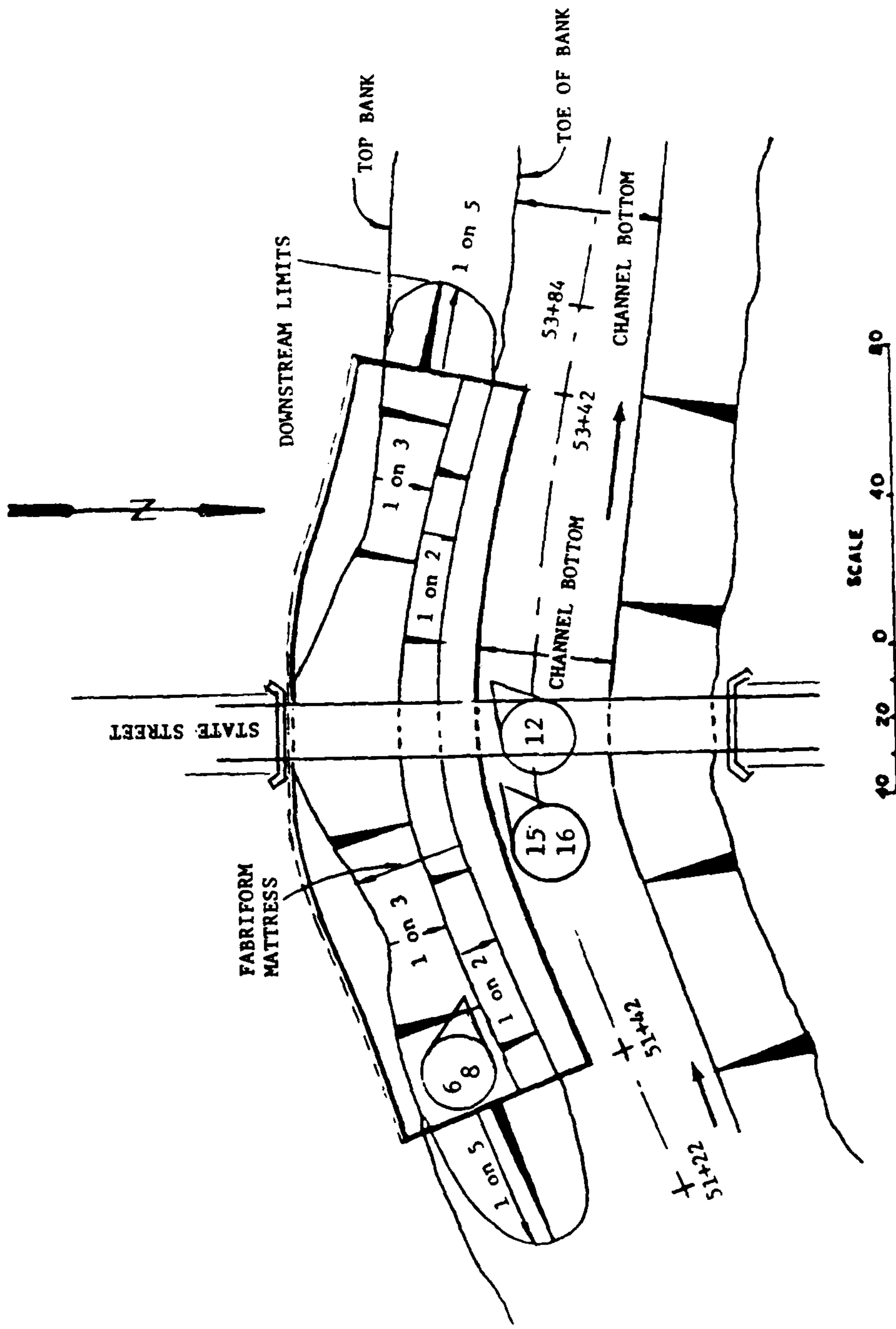


Figure 4. 102 River (East Fork) at Bedford, Iowa. Plan view of Fabriform revetment of left bank at State Street Bridge (sta 52+42). Orientation of field-of-view and location from which photograph was taken are represented by ◯▷ for Figures 6, 8, 12, 15, and 16.

ITEM 15-4

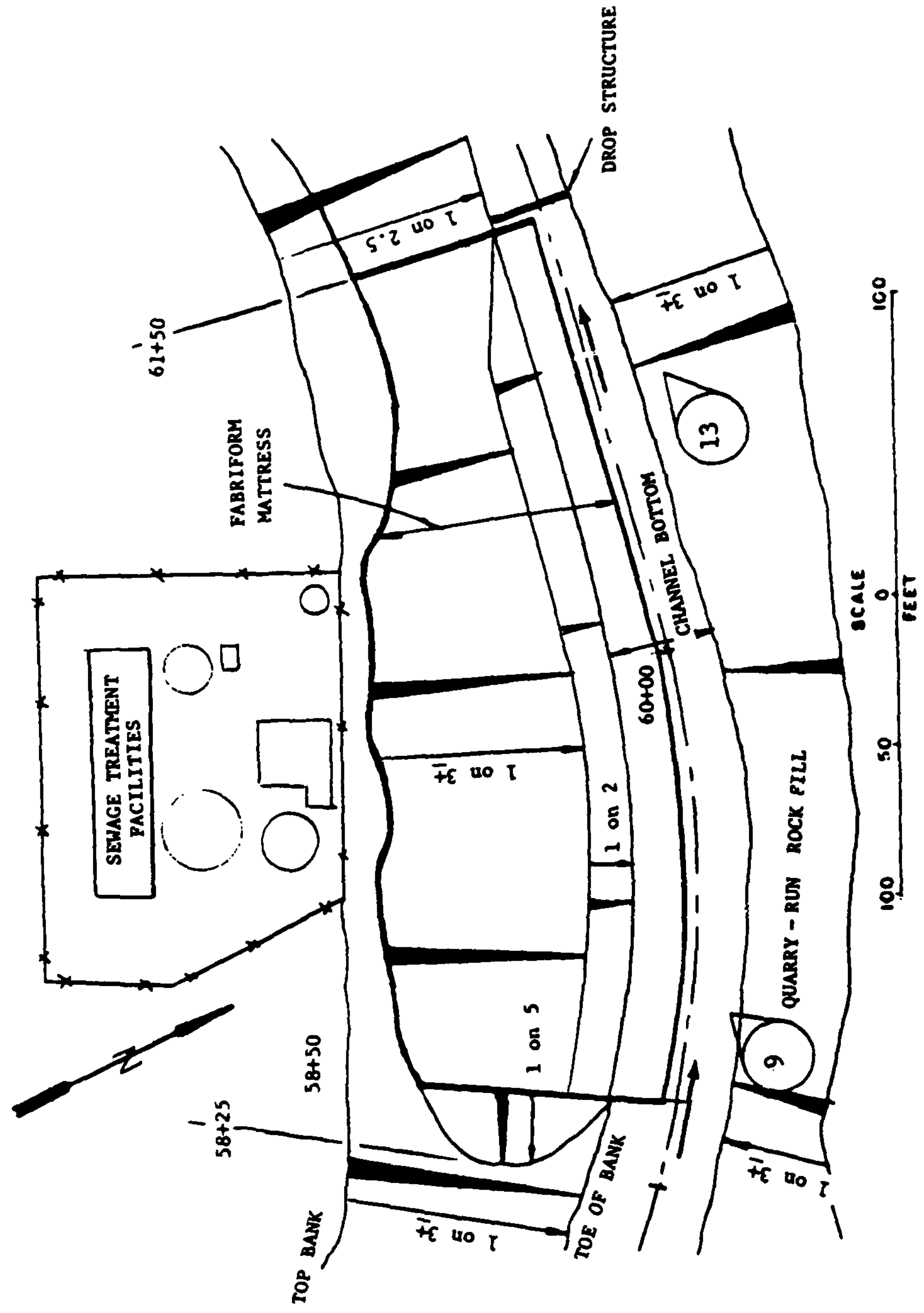


Figure 5. 102 River (East Fork) at Bedford, Iowa. Plan view of Fabriform revetment at Bedford Sewage Treatment Plant (sta 60+00). Orientation of field-of-view and location from which photograph was taken are represented by ◯▷ for Figures 9 and 13.

Figure 6. 102 River (East Fork) at Bedford, Iowa. Quarry-run riprap was used to shape the bank prior to placement of Fabriform. View is of the left bank of the State Street Bridge.

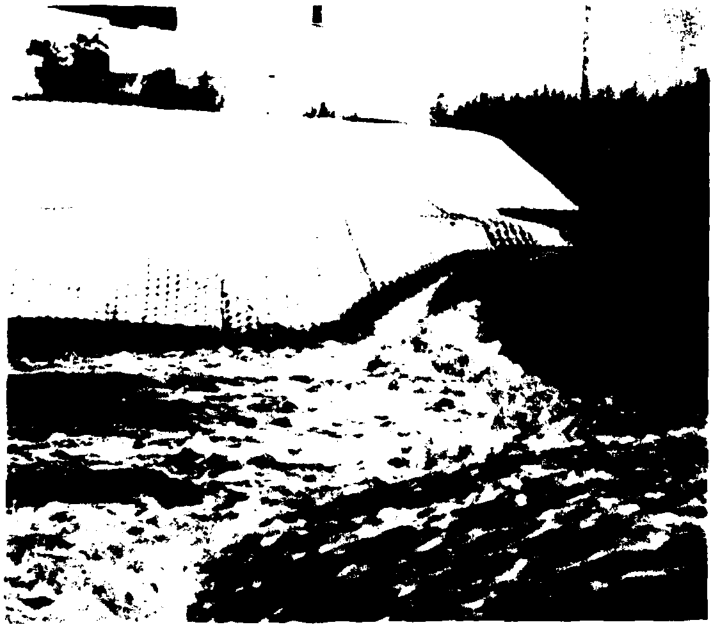


Figure 7. 102 River (East Fork) at Bedford, Iowa. Completed Fabriform revetment at Bedford Water Treatment Plant. Note low-head dam in foreground.

Figure 8. 102 River (East Fork) at Bedford, Iowa. Completed Fabriform revetment at State Street Bridge.

Figure 9. 102 River(East Fork) at Bedford, Iowa. Completed Fabriform revetment at Bedford Sewage Treatment Plant.

ITEM 15-7

Figure 10. 102 River (East Fork) at Bedford, Iowa. High flows undercut the Fabriform mattresses at several locations resulting in parts of the revetment breaking off and being swept into the center of the channel.

Figure 11. 102 River (East Fork) at Bedford, Iowa. High flows undercut the Fabriform mattress at the Bedford Water Treatment Plant resulting in subsidence of a section of the revetment.

ITEM 15-8

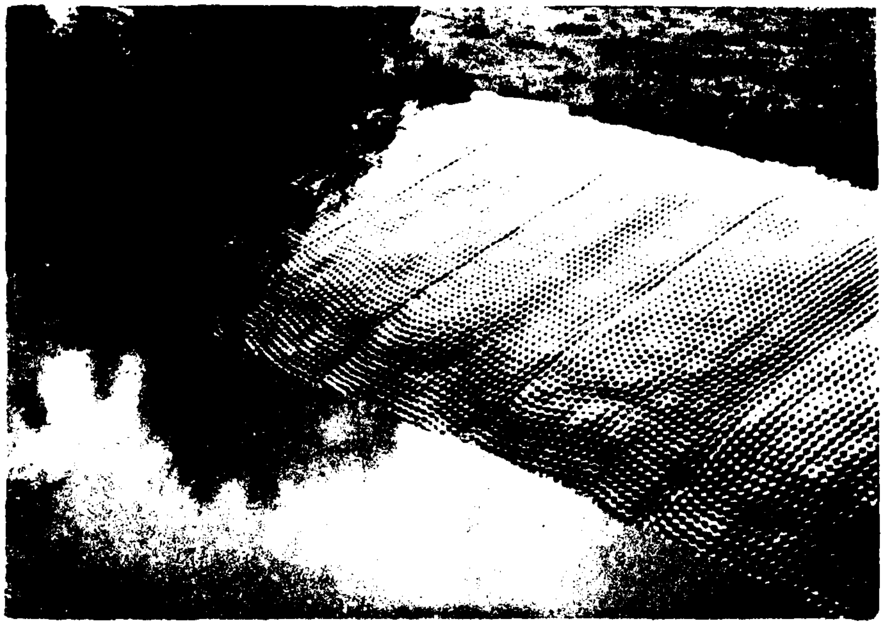


Figure 12. 102 River (East Fork) at Bedford, Iowa. At the time of the 1978 inspection visit, most of the Fabriform revetment was intact. This view shows the mattress downstream from the State Street Bridge.

Figure 13. 102 River (East Fork) at Bedford, Iowa. Failure of drop structure at sta 61+60 (March 1979). Note damaged Fabriform mattress in left portion of view.

Figure 14. 102 River (East Fork) at Bedford, Iowa. An inspection at the Bedford Water Treatment Plant in March 1979 indicated that a large cavity had formed under the mattress, which collapsed.

Figure 15. 102 River (East Fork) at Bedford, Iowa. View of damaged Fabriform revetment upstream of the State Street Bridge.

ITEM 15-10

Figure 16. 102 River (East Fork) at Bedford, Iowa. View of damaged Fabriform revetment upstream of the State Street Bridge.

GERING DRAIN
NEAR GERING, NEBRASKA

Streambank Erosion Control Evaluation and Demonstration Act of 1974
Section 32 Program - Work Unit 2

EVALUATION OF EXISTING BANK PROTECTION WORKS

(1) Location

Stream Gering Drain (fencing) River Mile ______ Side ______

Local Vicinity ______ Lat N41°45' Long W 103°45'

At/Nr City Gering County Scottsbluff State NE Cong Dist ______

CE Office Symbol MRO Responsible Agency Corps of Engineers

Site Map Sources Omaha District, Corps of Engineers

Land Use Agricultural

(2) Hydrology at or Near Site

Stage Range ______ to ______ ft; Period of Record 19__ to 19__.

Discharge Range ______ to ______ cfs; Velocity Range ______ to ______ fps

Sediment Range ______ to ______ tpd; Period of Record 19__ to 19__.

Bank-full Stage ______ ft; Flow 6,700 cfs; Average Recurrence Interval 50 yr

Bank-full Flow Velocity: Average 6 fps; Near Bank ______ fps

Comments ______

(3) Geology and Soil Properties

Bank (USCS) Sandy silt(ML) silty sand(SM) Bed (USCS) Silty sand over Brule siltstone

Data Sources Corps of Engineers test borings.

Groundwater Bank Seepage Increased during irrigation seasons.

Overbank Drainage Controlled

Comments ______

(4) Construction of Protection

Need for Protection Rapid enlargement of drains and waterways due to erosion.

Erosion Causative Agents Return water from irrigation in easily erodible soils and runoff from localized intense thunderstorms.

Protection Techniques Fencing and rock sills.

General Design Rock sills with 2 ft. head loss and double row of stone-filled fence between them.

Project Length ______ ft; Construction Cost $ ______ Mo/Yr Completed 4/69

(5) Maintenance

Experienced Flows (Stage, cfs, Date) ______

Repairs and Costs (Item, Cost, Data) ______

Comments: ______

(6) Performance Observations and Summary

Monitoring Program Visual inspection, surveys, and photographs

Documentation Sources Omaha District, Corps of Engineers

Project Effect on Stream Regime Reversed degradation trend and stabilized the channel.

Project Effect on Environment Permitted revegetation of streambanks and increased wildlife habitat.

Successful Aspects Stabilized the channel.

Unsuccessful Aspects ______

General Evaluation ______

Recommendations ______

(7) Additional Information, Comments, and Summary

Map No. 16.

Attached Items:

16-1 Project summary
16-2 Project location
16-3 Project plan
16-(4-6) Photographs

Gering Drain at Gering, Nebraska

The Gering Valley is located in Scottsbluff County, Nebraska, in the western extremity of the Nebraska Panhandle. The valley originates at the foot of Roubadeau Pass and extends eastward about 16 miles, terminating at the North Platte River. The Gering Valley is a relatively steep, heavily irrigated and intensively cultivated area ringed on three sides by a U-shaped range of bluffs. These bluffs extend about 600 feet above the valley with a maximum elevation of approximately 4,800 feet, m.s.l. The valley is drained by a man-made waterway and drainage canal known as Gering Drain, which has its outlet at the North Platte River in the northeast corner of the valley, Figure 1. The Drain originates below the Fort Laramie Canal at the foot of Roubadeau Pass and flows eastward along land lines, draining an area of approximately 84 square miles. A lateral system of drains collects irrigation return flows and rainfall runoff and connects the Gering Drain with the mouths of the hill canyons. The town of Gering, Nebraska, the only community in the valley, is located in the northeast portion of the valley, south of and across the North Platte River from the larger city of Scottsbluff. The entire valley is irrigated by water from the Fort Laramie Canal, the Gering Canal, and the Central Canal.

Originally the Gering Valley was poorly drained, with no well-defined watercourse to convey water to the North Platte River. Runoff spread over a wide area and was largely dissipated through infiltration and evaporation, with only a relatively small portion finding its way overland to the river. With the installation of extensive irrigation in the 1920's, the additional water brought into the valley quickly created drainage problems. In order to lower the high water table and to provide a means for disposal of irrigation return flows, the Gering Drain and its several lateral drains were constructed in 1925 and shortly thereafter. These drains obviously were not designed to convey runoff from intense storms, for they were constructed with relatively small channels,

ITEM 16-1
(Sheet 1 of 5)

generally about 10 feet deep and with about a 10-foot bottom width. However, these drains provided a means for collection and concentration of storm runoff, and new problems developed. Where storm runoff previously had spread out over considerable areas in the upper and central valley, it now concentrated in and along the drains and was transported to the lower valley where a new flood problem was created. The soils in which the drains were dug are primarily silty sands which have little cohesive property and are easily eroded. They are underlain by a widespread formation of Brule siltstone which, although relatively resistant in comparison to the surface soils, is subject to erosion when exposed to prolonged flow of water. As a consequence, the drains continually enlarged through both erosion of the banks and degradation of the bottoms.

Local interests, to the extent of their limited resources, had attempted to control the destructive effects of erosion. Generally, their efforts had been concentrated on localized control of erosion which threatened destruction of irrigation structures and road bridges. These efforts consisted primarily of construction of relatively small sheet-pile drop structures. However, they also constructed one major control structure known as the Ostenburg Chute, a large concrete flume-type drop structure in the Gering Drain, which controlled a vertical drop of about 19 feet at the edge of the North Platte River floodplain. The former location of this structure is shown in Figure 1. The Ostenburg Chute was completely destroyed in 1958 and the drainage channels upstream were made vulnerable to continuing further degradation and enlargement.

At the time of preparation of the report contained in Senate Document No. 139 (1953) flooding was the major problem, although erosion was also recognized as a serious problem and improvements recommended in the report were aimed at correction of both problems. Although erosion and degradation had already enlarged the upper and central valley drains to major proportions, the Ostenburg Chute was still in place and was

effectively controlling the major grade drop at that location. Channel capacity in the vicinity of the chute had been stabilized at about 1,800 c.f.s. Flood flows from the hill areas were contained in the greatly enlarged Gering Drain to the vicinity of State Highway 29, with overflow commencing at that point and subjecting about 4,000 acres downstream to periodic inundation. Prior to 1953, 16 floods had occurred in the area mentioned above. The first record of flooding was in 1931, or shortly after construction of the Gering Drain and its laterals. Average annual flood damages were estimated (1952 price levels) at $52,200, based on the then existing state of economic development, the then existing physical characteristics of the valley and channels, and discharge probabilities computed from historical records. The report recognized that the grade stabilization effected by the Ostenburg Chute was of major importance, that the limited discharge capacity of the chute made it vulnerable to destruction by floods, and that additional chute capacity was desirable. It recognized that erosion in the drain upstream from the chute was a threat to irrigation structures and bridges. It recognized also that seepage from irrigation canals appreciably raised the groundwater table, causing intensified erosion of the banks of the Gering Drain and its laterals and that land practices were such that runoff from the fields was rapidly adding to the flood potential and bank erosion problem.

The USGS has maintained a stream gaging station below Highway 86 from 1931 to the present. Maximum discharge observed at the USGS gage was 8,000 c.f.s. in June 1947 and June 1958. During the storm of June 1958, the Ostenburg Chute's capacity (about 1,800 c.f.s.) was exceeded and it was destroyed after being overtopped and flanked. Following failure of the structure, a wave of channel degradation moved rapidly upstream and, if left unchecked, would have led to damage to or failure of other existing channel control structures.

A comprehensive plan of protection for the entire valley was developed jointly by the Soil Conservation Service, the Corps of Engineers,

and Local Interests. The plan included the following work by the Soil Conservation Service: (a) a system of reservoirs at the edges of the valley to reduce channel and overland flow, permitting substantial savings in the design of downstream erosion control methods; (b) land treatment including a conservation cropping system, bench leveling, farm diversions, and proper use of irrigation water; and (c) channel improvement on two tributary drains, 320 on-farm stabilization structures, and four large channel stabilization structures. Work to be performed by the Corps of Engineers included: (a) construct approximately 120,000 lineal feet of earth barriers along the Main Drain and its tributaries plus grassed waterways with approximately 40,000 lineal feet of concrete interceptor channels landward of the earth barriers for conveying surface runoff to the interior drainage structures; (b) approximately 50 interior drainage structures designed to pass the surface runoff from the grassed waterways into the drains without eroding the banks; and (c) a total of 17 rock sills with parallel fencing, 4,000 lineal feet of rock lining, 24 concrete drop structures, and 7 culvert drops on the Main Drain and its tributary drains to provide grade stabilization. Following completion of the above construction, Local Interests are to conduct a systematic continuing construction program to provide low sloping berms along the channel bottoms between structures and to promote vegetative growth where it is needed for preventing bank erosion.

In reaches where the channels are wide and not too steep, a series of low rock sills between two parallel rows of double wire fencing, as shown in Figures 2, 3, and 4, was found to be the most adaptable solution. The rock sills are spaced at approximately 500 feet (depending on the bed slope) and create a head loss of about 2 feet each. The two parallel rows of fencing were placed along the desired channel alinement and filled with rock or hay bales. Earthen groins were spaced at about 100-foot intervals across the berm area to prevent flow from developing behind the fencing, Figure 2. The fencing was generally placed in a double row and the space between was filled with stone or hay bales,

Figures 3 and 4. The fencing served as a permeable revetment to trap silt to reestablish a smoother bank alinement. In areas where a possibility of further degradation existed, the stone was placed in the fencing so that it would migrate out of the fencing and armor the toe as degradation occurred, Figure 5. A minor problem developed in some areas when the bottom of the fencing was deformed so that the stone could not move downward freely.

The rock sills, fencing, and earthen groins were constructed as planned and willows were also planted in the berm areas to induce siltation. All structures placed during initial construction phases have functioned as planned. Following the initial construction which was completed in 1969, downstream degradation required the construction of four additional rock sills and fencing in 1973 at the lower end of the project. The channel below these structures has continued to degrade slowly. Degradation below rock sill 1D is between 4 and 5 feet and has undercut the fencing. This may eventually cause failure of the fencing and erosion of the channel banks.

It appears that where the channel degradation has been controlled, the fencing has performed very well in trapping sediments and contributed to the establishment of stabilized flow lines and revegetation of the berm areas, Figure 6.

ITEM 16-1
(Sheet 5 of 5)

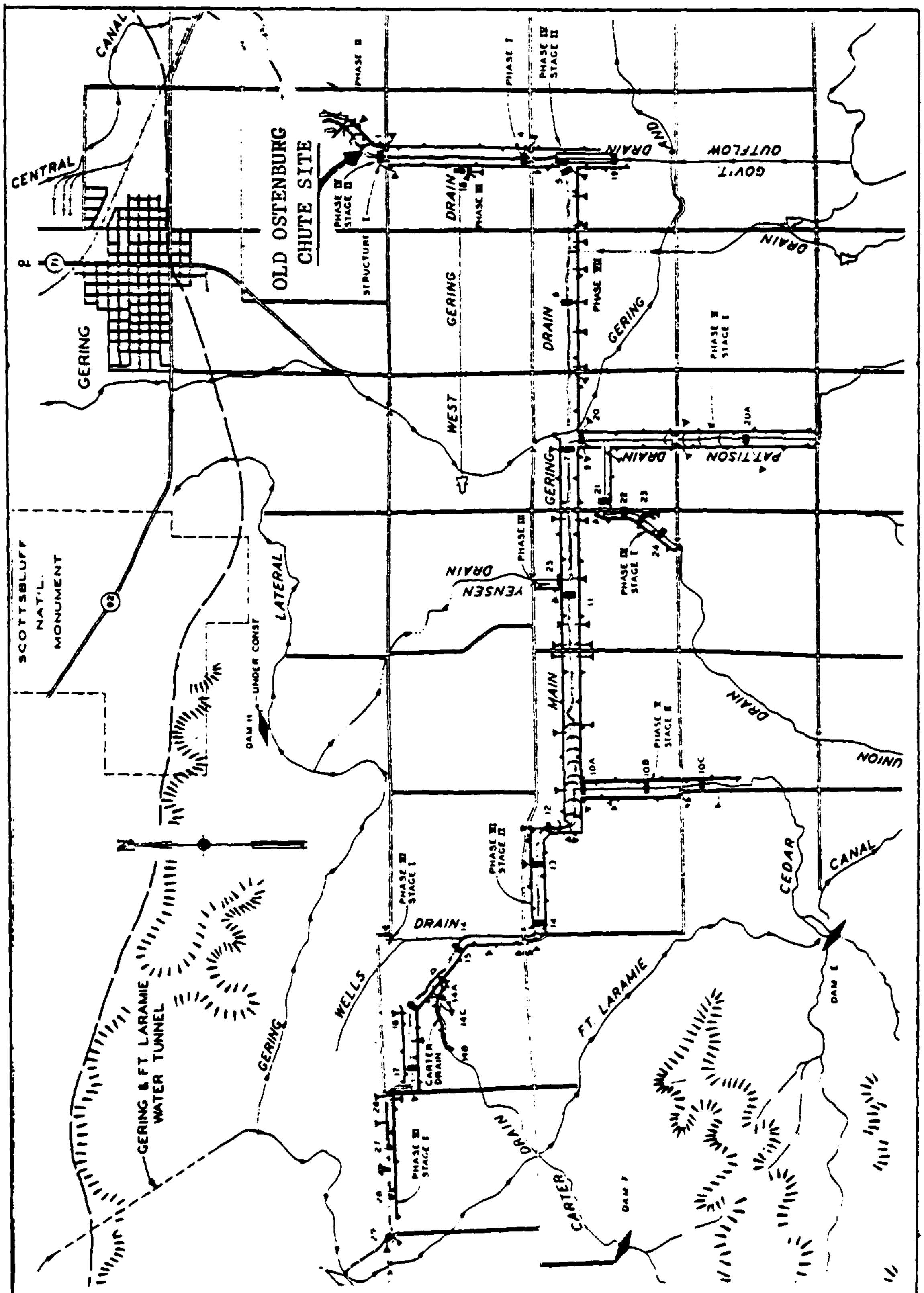


Figure 1. Gering Drain near Gering, Nebraska

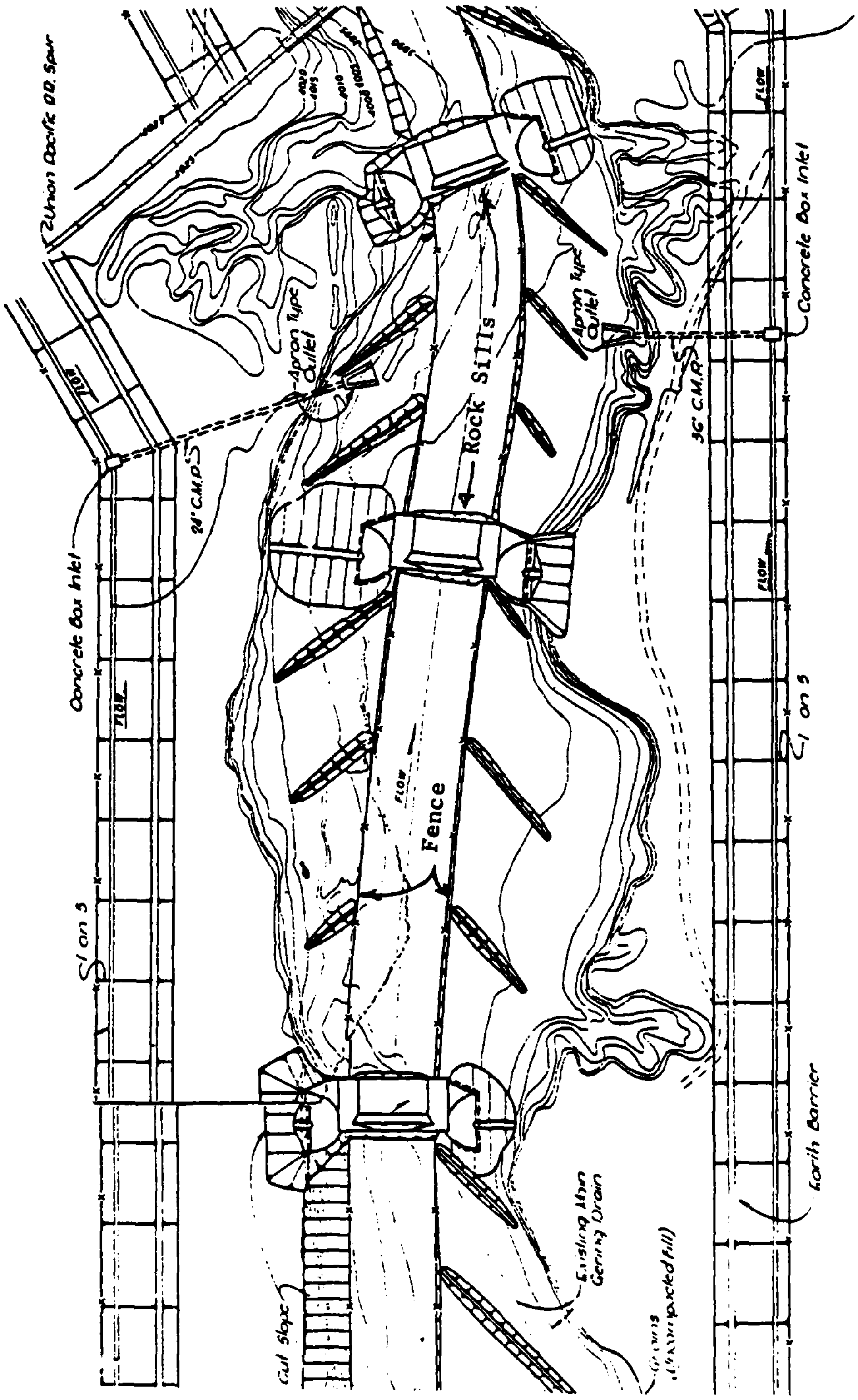


Figure 2. Gering Drain near Gering, Nebraska. Typical layout of low rock sills and fencing.

Figure 3. Gering Drain near Gering, Nebraska. View of double-row fence filled with stone.

Figure 4. Gering Drain near Gering, Nebraska. View of double-row fence filled with hay bales

Figure 5. Gering Drain near Gering, Nebraska. Stone filled double-row fence and stabilized channel banks

ITEM 16-5

Figure 6. Gering Drain near Gering, Nebraska. View of stone-filled fence with stone moving down the bank

PLUM CREEK
NEAR DENVER, COLORADO

Streambank Erosion Control Evaluation and Demonstration Act of 1974
Section 32 Program - Work Unit 2

EVALUATION OF EXISTING BANK PROTECTION WORKS

(1) Location

Stream Plum Creek River Mile ______ Side Right

Local Vicinity ______ Lat N39°45' Long W104°45'

At/Nr City Denver County Douglas State CO Cong Dist 1

CE Office Symbol MRO Responsible Agency Corps of Engineers

Site Map Sources Omaha District, Corps of Engineers

Land Use Agricultural and developing urbanization

(2) Hydrology at or Near Site

Stage Range NA to ______ ft; Period of Record 19__ to 19__.

Discharge Range NA to ______ cfs; Velocity Range ______ to ______ fps

Sediment Range NA to ______ tpd; Period of Record 19__ to 19__.

Bank-full Stage ______ ft; Flow ______ cfs; Average Recurrence Interval ______ yr

Bank-full Flow Velocity Average ______ fps; Near Bank ______ fps

Comments No hydrologic measurements available.

(3) Geology and Soil Properties

Bank (USCS) Lean clays, silty sand Bed (USCS) Lean clays, silty sand

Data Sources Corps of Engineers test borings.

Groundwater Bank Seepage ______

Overbank Drainage ______

Comments ______

(4) Construction of Protection

Need for Protection Flooding caused serious bank erosion, exposing 40-inch waterline to Aurora, Colorado

Erosion Causative Agents High flows.

Protection Techniques Reestablish and stabilize bank line. (Fencing)

General Design RR Rail posts set 4'8" in ground on 8'centers and cable connected, wire mesh tied on, rock dike tiebacks to high bank.

Project Length 1,000 ft; Construction Cost $ 49,000 Mo/Yr Completed 6/70

(5) Maintenance

Experienced Flows (Stage, cfs, Date) ____

Repairs and Costs (Item, Cost, Data) ____

Comments: ____

(6) Performance Observations and Summary

Monitoring Program Visual inspections.

Documentation Sources Corps of Engineers, Omaha District

Project Effect on Stream Regime Stabilized bank line.

Project Effect on Environment ____

Successful Aspects Has stabilized the banks and prevented damage to waterline

Unsuccessful Aspects ____

General Evaluation Overall the project has performed as designed.

Recommendations ____

(7) Additional Information, Comments, and Summary

Map No. 17.

Attached Items:

17-1 Project summary and location
17-2 Project plan and cross section
17-3 Fencing details
17-4 "

Plum Creek, Douglas County, Colorado

Plum Creek is a right bank tributary of the South Platte River in Douglas County, Colorado, south of Littleton, Figure 1. Plum Creek rises approximately 40 miles south of Denver and flows in a northerly direction. The drainage area is 324 square miles and begins in the foothills of the Rocky Mountains. Most of the drainage area is in an area of gently sloping grasslands.

The city of Aurora, Colorado, has a 40-inch steel waterline under the streambed about 4 miles upstream from the mouth of Plum Creek. In June 1965 a flood on Plum Creek eroded the streambanks in the vicinity of the water supply line crossing, badly damaged the line, and caused loss of water supply to the city. Subsequent to this flood, the line was replaced and encased in concrete for the full width of the enlarged Plum Creek channel which was more than 200 feet wide at that time. In May 1968, flooding on Plum Creek was again eroding the banks and endangered the water supply line. The city of Aurora requested assistance from the Corps of Engineers under Section 14 of the Flood Control Act of 1946 in controlling the erosion. A plan of protection was developed using a combination of stone-fill dikes, stone-fill revetments, and a single row of fencing to act as a permeable revetment, Figure 2. The project consists of 250 linear feet of stone-fill revetment, 140 linear feet of stone-fill root, 320 linear feet of stone spur dikes, 990 linear feet of erosion control fencing, and 100 linear feet of stone-fill bank protection. Construction was completed in June 1970. The fencing portion of the project was selected as an existing site for the Section 32 program.

The 990 linear feet of erosion control fencing was constructed using salvaged railroad rail as posts set on an 8-ft spacing. The posts and fencing were stabilized with three 9/16-in. cables, which traversed the fence line at the bottom, middle, and top of the posts. The cables were passed through holes burned through the rails. The fencing was then

attached to the rails and cables with two strands of twisted No. 12 galvanized-steel wire, Figures 3 and 4. The fence fabric was 2- x 4-in. V-mesh No. 12 galvanized-steel wire.

The project has functioned very well. Figure 5 shows the landward side of the fence revetment and one of the stone spur dikes. Figure 6 shows the bank stabilized and willow growth established.

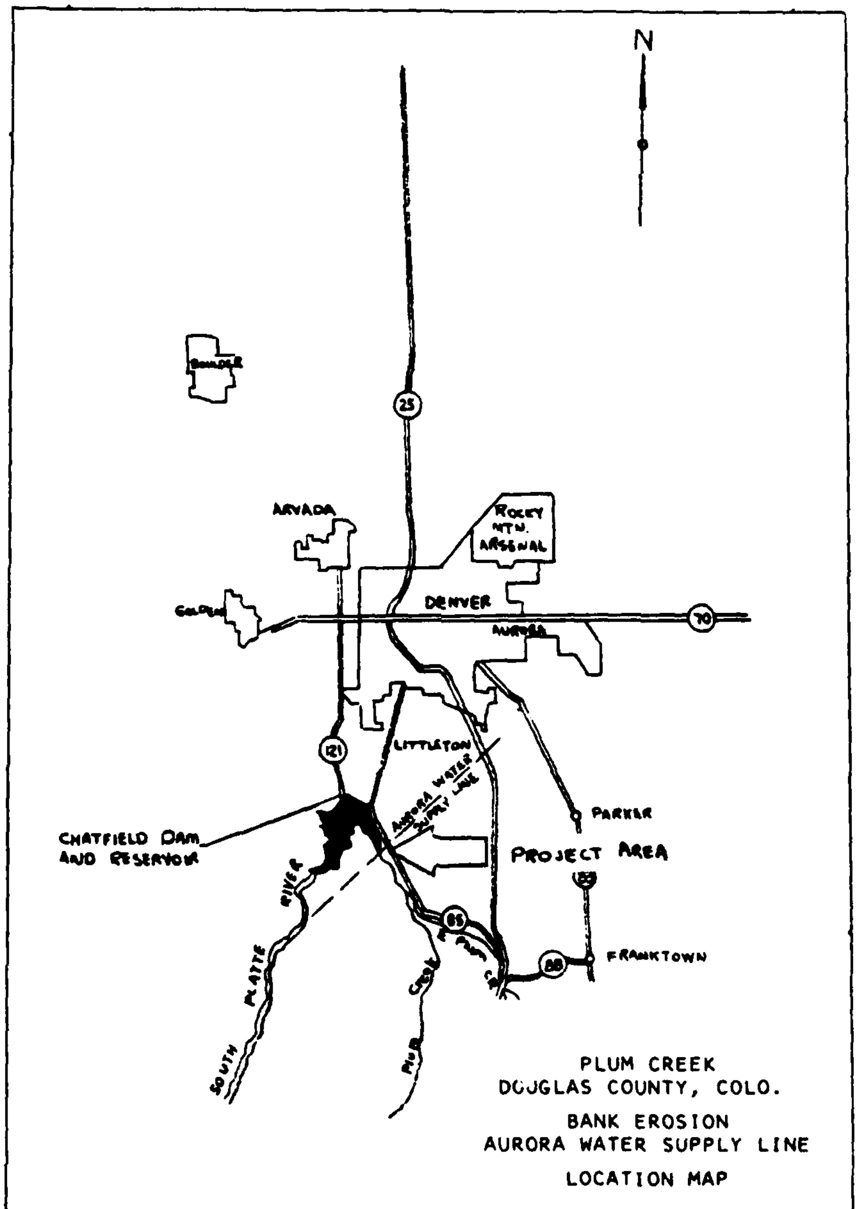


Figure 1.

ITEM 17-1
(Sheet 2 of 2)

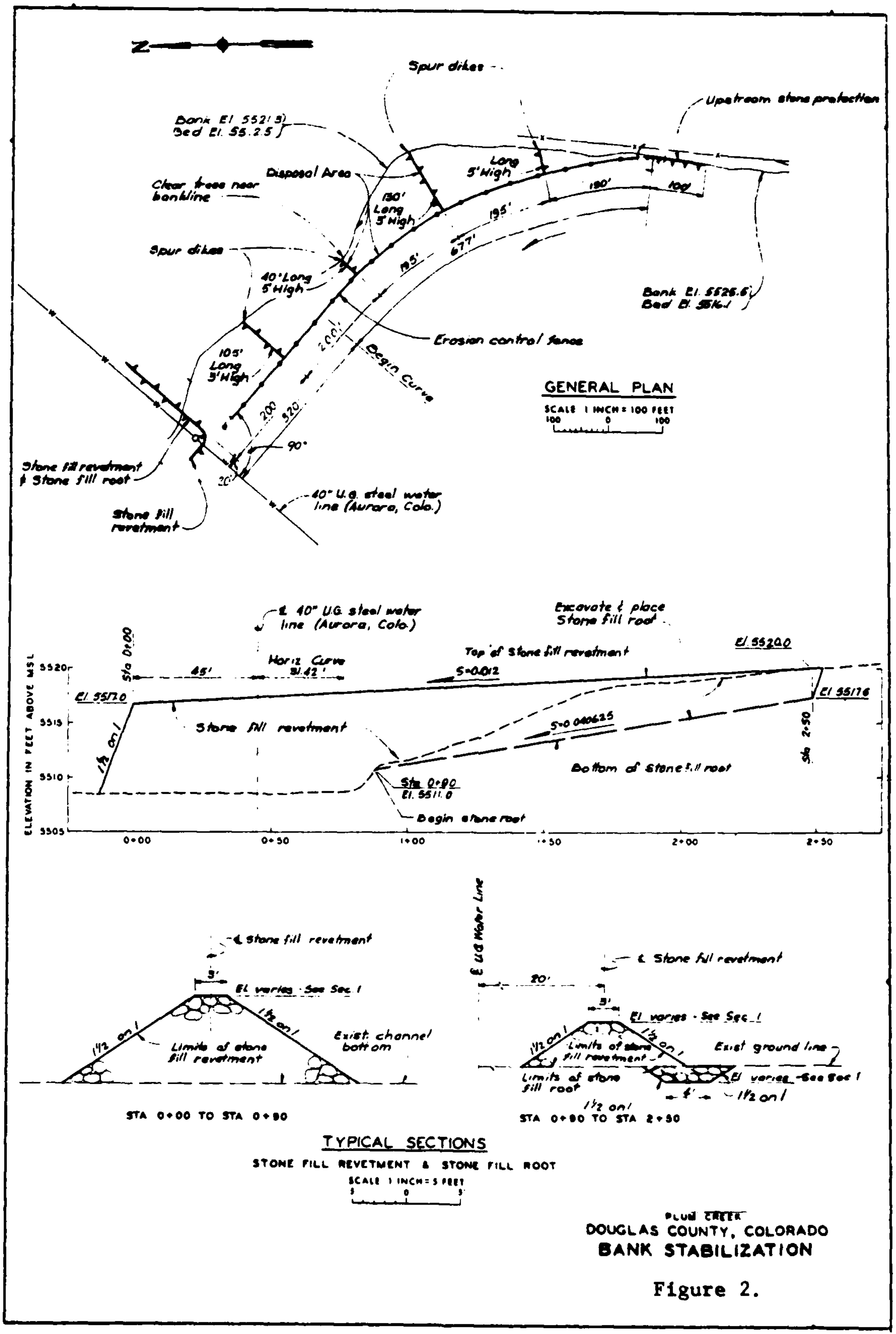
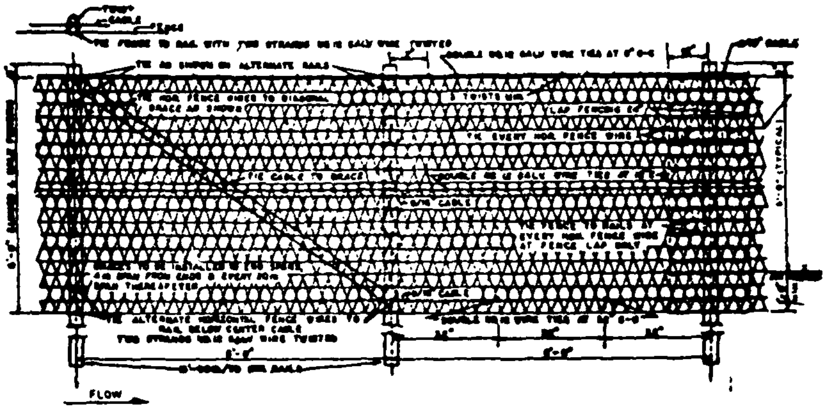


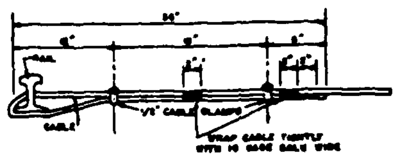
Figure 2.

ITEM 17-2



DETAIL OF FENCING-FASTENINGS

SCALE $\frac{3}{8}$ INCH = 1 FOOT



DETAIL OF CABLE FASTENINGS

SCALE 1" = 1'

NOTES:

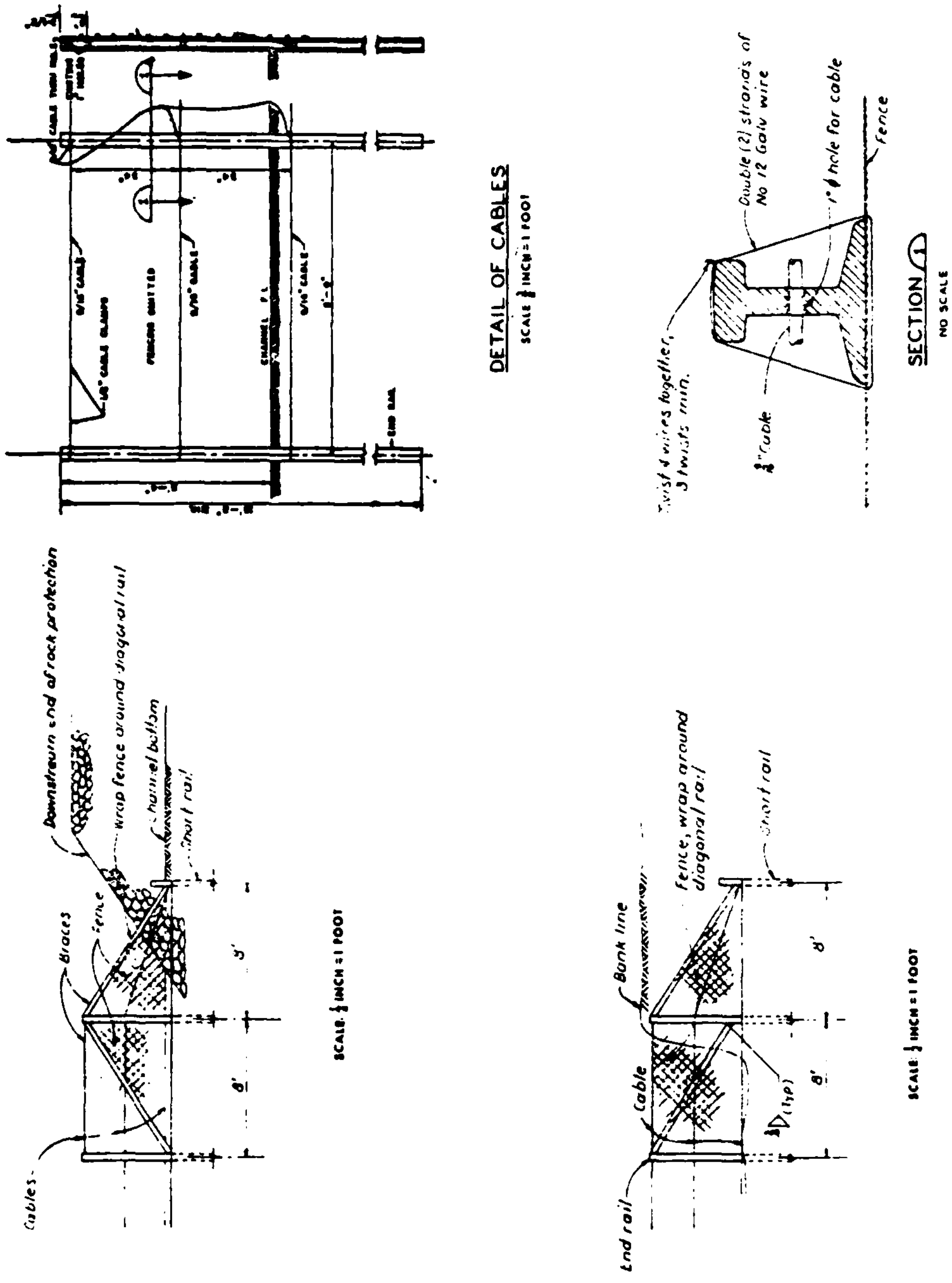
1. Used cable & rails may be used as approved by the contracting officer.
2. Diagonal braces same section as posts. Weld brace to posts.
3. Cable diameter may vary $\pm\frac{1}{16}$".

PLUM CREEK

DOUGLAS COUNTY, COLORADO

BANK STABILIZATION

Figure 3



PLUM CREEK
DOUGLAS COUNTY, COLORADO
BANK STABILIZATION

Figure 4

Figure 5. Plum Creek, Douglas County, Colorado. Landward side of fence revetment with tieback.

Figure 6. Plum Creek, Douglas County, Colorado. Willow growth becoming reestablished.

ITEM 17-5

GERING DRAIN
GERING, NEBRASKA

Streambank Erosion Control Evaluation and Demonstration Act of 1974
Section 32 Program - Work Unit 2

EVALUATION OF EXISTING BANK PROTECTION WORKS

(1) Location

Stream Gering Drain (rock sills) River Mile ______ Side ______

Local Vicinity ______ Lat N41°45' Long W103°45'

At/Nr City Gering County Scottsbluff State NE Cong Dist ______

CE Office Symbol MRO Responsible Agency Corps of Engineers

Site Map Sources Omaha District, Corps of Engineers

Land Use Agricultural

(2) Hydrology at or Near Site

Stage Range ______ to ______ ft; Period of Record 19__ to 19__.

Discharge Range ______ to ______ cfs; Velocity Range ______ to ______ fps

Sediment Range ______ to ______ tpd; Period of Record 19__ to 19__.

Bank-full Stage ______ ft; Flow 6,700 cfs; Average Recurrence Interval 50 yr

Bank-full Flow Velocity: Average 6 fps; Near Bank ______ fps

Comments ______

(3) Geology and Soil Properties

Bank (USCS) Sandy silt(ML),silty sand(SM) Bed (USCS) Silty sand over Brule siltstone

Data Sources Corps of Engineers test borings.

Groundwater Bank Seepage Increased during irrigation seasons.

Overbank Drainage Controlled

Comments ______

(4) Construction of Protection

Need for Protection Rapid enlargement of drains and waterways due to erosion.

Erosion Causative Agents Return water from irrigation in easily erodible soils and runoff from localized intense thunderstorms.

Protection Techniques Fencing and rock sills.

General Design Rock sills with 2 ft head loss and double row of stone-filled fence between them.

Project Length ______ ft; Construction Cost $ ______ Mo/Yr Completed 4/69

(5) Maintenance

Experienced Flows (Stage, cfs, Date) ______

Repairs and Costs (Item, Cost, Data) ______

Comments: ______

(6) Performance Observations and Summary

Monitoring Program Visual inspections, surveys, and photographs

Documentation Sources Omaha District, Corps of Engineers

Project Effect on Stream Regime Reversed degradation trend and stabilized the channel.

Project Effect on Environment Permitted revegetation of streambanks and increased wildlife habitat.

Successful Aspects Stabilized the channels.

Unsuccessful Aspects ______

General Evaluation ______

Recommendations ______

(7) Additional Information, Comments, and Summary

Map No. 18.

Attached Items:

18-1 Project summary
18-2 Project location
18-3 Typical layout
18-4 Photographs

Gering Drain at Gering, Nebraska

The Gering Valley is located in Scottsbluff County, Nebraska, in the western extremity of the Nebraska Panhandle. The valley originates at the foot of Roubadeau Pass and extends eastward about 16 miles, terminating at the North Platte River. The Gering Valley is a relatively steep, heavily irrigated and intensively cultivated area ringed on three sides by a U-shaped range of bluffs. These bluffs extend about 600 feet above the valley with a maximum elevation of approximately 4,800 feet, m.s.l. The valley is drained by a man-made waterway and drainage canal known as Gering Drain, which has its outlet at the North Platte River in the northeast corner of the valley, Figure 1. The Drain originates below the Fort Laramie Canal at the foot of Roubadeau Pass and flows eastward along land lines, draining an area of approximately 84 square miles. A lateral system of drains collects irrigation return flows and rainfall runoff and connects the Gering Drain with the mouths of the hill canyons. The town of Gering, Nebraska, the only community in the valley, is located in the northeast portion of the valley, south of and across the North Platte River from the larger city of Scottsbluff. The entire valley is irrigated by water from the Fort Laramie Canal, the Gering Canal, and the Central Canal.

Originally the Gering Valley was poorly drained, with no well-defined watercourse to convey water to the North Platte River. Runoff spread over a wide area and was largely dissipated through infiltration and evaporation, with only a relatively small portion finding its way overland to the river. With the installation of extensive irrigation in the 1920's, the additional water brought into the valley quickly created drainage problems. In order to lower the high water table and to provide a means for disposal of irrigation return flows, the Gering Drain and its several lateral drains were constructed in 1925 and shortly thereafter. These drains obviously were not designed to convey runoff from intense storms, for they were constructed with relatively small channels,

generally about 10 feet deep and with about a 10-foot bottom width. However, these drains provided a means for collection and concentration of storm runoff, and new problems developed. Where storm runoff previously had spread out over considerable areas in the upper and central valley, it now concentrated in and along the drains and was transported to the lower valley where a new flood problem was created. The soils in which the drains were dug are primarily silty sands which have little cohesive property and are easily eroded. They are underlain by a widespread formation of Brule siltstone which, although relatively resistant in comparison to the surface soils, is subject to erosion when exposed to prolonged flow of water. As a consequence, the drains continually enlarged through both erosion of the banks and degradation of the bottoms.

Local interests, to the extent of their limited resources, had attempted to control the destructive effects of erosion. Generally, their efforts had been concentrated on localized control of erosion which threatened destruction of irrigation structures and road bridges. These efforts consisted primarily of construction of relatively small sheet-pile drop structures. However, they also constructed one major control structure known as the Ostenburg Chute, a large concrete flume-type drop structure in the Gering Drain, which controlled a vertical drop of about 19 feet at the edge of the North Platte River floodplain. The former location of this structure is shown in Figure 1. The Ostenburg Chute was completely destroyed in 1958 and the drainage channels upstream were made vulnerable to continuing further degradation and enlargement.

At the time of preparation of the report contained in Senate Document No. 139 (1953) flooding was the major problem, although erosion was also recognized as a serious problem and improvements recommended in the report were aimed at correction of both problems. Although erosion and degradation had already enlarged the upper and central valley drains to major proportions, the Ostenburg Chute was still in place and was

AD-A121 140 THE STREAMBANK EROSION CONTROL EVALUATION AND DEMONSTRATION ACT OF 1974 S..(U) ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG MS HYDRA.. 3/3

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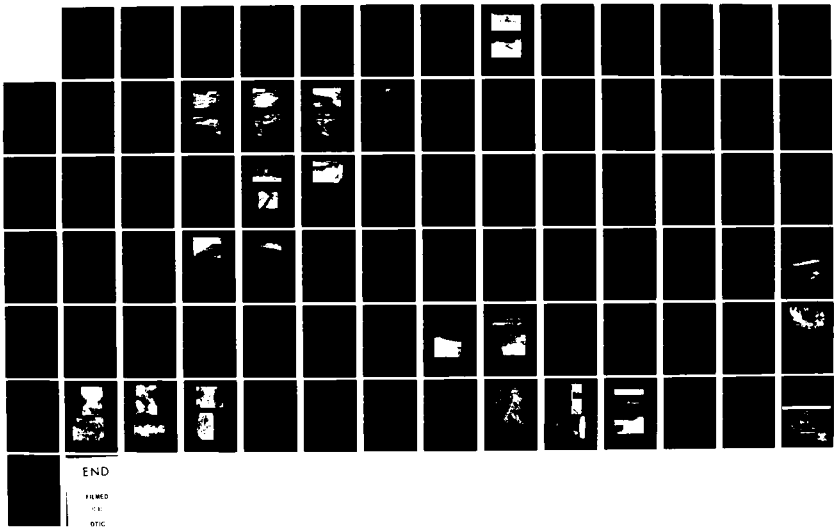

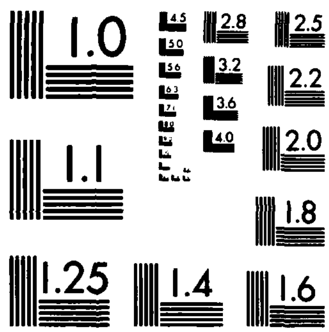

MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

effectively controlling the major grade drop at that location. Channel capacity in the vicinity of the chute had been stabilized at about 1,800 c.f.s. Flood flows from the hill areas were contained in the greatly enlarged Gering Drain to the vicinity of State Highway 29, with overflow commencing at that point and subjecting about 4,000 acres downstream to periodic inundation. Prior to 1953, 16 floods had occurred in the area mentioned above. The first record of flooding was in 1931, or shortly after construction of the Gering Drain and its laterals. Average annual flood damages were estimated (1952 price levels) at $52,200, based on the then existing state of economic development, the then existing physical characteristics of the valley and channels, and discharge probabilities computed from historical records. The report recognized that the grade stabilization effected by the Ostenburg Chute was of major importance, that the limited discharge capacity of the chute made it vulnerable to destruction by floods, and that additional chute capacity was desirable. It recognized that erosion in the drain upstream from the chute was a threat to irrigation structures and bridges. It recognized also that seepage from irrigation canals appreciably raised the groundwater table, causing intensified erosion of the banks of the Gering Drain and its laterals and that land practices were such that runoff from the fields was rapidly adding to the flood potential and bank erosion problem.

The USGS has maintained a stream gaging station below Highway 86 from 1931 to the present. Maximum discharge observed at the USGS gage was 8,000 c.f.s. in June 1947 and June 1958. During the storm of June 1958, the Ostenburg Chute's capacity (about 1,800 c.f.s.) was exceeded and it was destroyed after being overtopped and flanked. Following failure of the structure, a wave of channel degradation moved rapidly upstream and, if left unchecked, would have led to damage to or failure of other existing channel control structures.

A comprehensive plan of protection for the entire valley was developed jointly by the Soil Conservation Service, the Corps of Engineers,

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(Sheet 3 of 6)

and Local Interests. The plan included the following work by the Soil Conservation Service: (a) a system of reservoirs at the edges of the valley to reduce channel and overland flow, permitting substantial savings in the design of downstream erosion control methods; (b) land treatment including a conservation cropping system, bench leveling, farm diversions, and proper use of irrigation water; and (c) channel improvement on two tributary drains, 320 on-farm stabilization structures, and four large channel stabilization structures. Work to be performed by the Corps of Engineers included: (a) construct approximately 120,000 lineal feet of earth barriers along the Main Drain and its tributaries plus grassed waterways with approximately 40,000 lineal feet of concrete interceptor channels landward of the earth barriers for conveying surface runoff to the interior drainage structures; (b) approximately 50 interior drainage structures designed to pass the surface runoff from the grassed waterways into the drains without eroding the banks; and (c) a total of 17 rock sills with parallel fencing, 4,000 lineal feet of rock lining, 24 concrete drop structures, and 7 culvert drops on the Main Drain and its tributary drains to provide grade stabilization. Following completion of the above construction, Local Interests are to conduct a systematic continuing construction program to provide low sloping berms along the channel bottoms between structures and to promote vegetative growth where it is needed for preventing bank erosion.

In reaches where the channels are wide and not too steep, a series of low rock sills between two parallel rows of double wire fencing, as shown in Figures 2, 3, and 4, was found to be the most adaptable solution. The rock sills are spaced at approximately 500 feet (depending on the bed slope) and create a head loss of about 2 feet each. The two parallel rows of fencing were placed along the desired channel alinement and filled with rock or hay bales. Earthen groins were spaced at about 100-foot intervals across the berm area to prevent flow from developing behind the fencing, Figure 2. The rock sills are designed to limit the 25-year discharge to a maximum velocity of 6 feet per second. The crest

widths are sized for a 50-year discharge and range between 17 and 120 feet. The top of the lateral embankments are set high enough to prevent overtopping by the 100-year discharge.

A model study of the rock sills was conducted during the summer of 1961 at the Hydraulics Laboratory of the University of California under the direction of Dr. H. A. Einstein. The purpose of the investigation was to determine the most desirable crest shape and to develop criteria for laying out the sills and fencing in natural channels. The tests were performed in a recirculating flume 60 feet long and 2 feet wide. Plastic pellets, which had a specific gravity of 1.055, were used to simulate the erodibility of the prototype material in Gering Valley. Rock, having a median size of 1 inch and a specific gravity of 2.75, was used in constructing the model sills. Test results were later checked using two other sizes of rock. Two basic studies were conducted as follows:

a. Various shaped structures, having the same height and volume of rock, were tested on a two-dimensional basis. The model tests included studies of both energy dissipation and scour characteristics. A weir crest, triangular in cross section, with a downstream horizontal apron was observed to have the most desirable shape of those investigated. This was the only shape where hydraulic action caused an eddy which satisfactorily maintained a slope of bed material against the downstream toe of the rock apron. This indicated minimum scour at the structure itself and would add protection against unravelling of the blanket. The horizontal rock apron also decreases the velocity near the bed and minimizes scour in the downstream channel. The initial tests indicated that it was not possible to lose enough head with the weir submerged; therefore, it was necessary to raise the crest high enough to induce critical depth. In the prototype the crest was actually constructed with a flattened top, 5 feet wide, to help avoid irregularities in the crest profile that might cause flow concentrations and subsequent crevassing.

b. A three-dimensional study of the rock sills was undertaken to determine the best layout at the ends of the sill, the limits to which the rock blanket should be extended, and the best arrangement of the wire fencing. Two sills were constructed in tandem to obtain the proper approach conditions to the downstream sill. Wire screen was used to simulate the proposed wire fencing and it worked very effectively. Bed material was deposited behind the wire screen and the majority of the flow was confined to the central channel. It was determined from the test results that 2.0 feet of head loss could be effectively controlled at each of the sills in the prototype with a median size rock of 18 in. (300 pounds) and a unit width discharge of 50 c.f.s. During the tests, design flows were exceeded to observe the behavior during ultimate failure of the rock sills. At a relatively high discharge (approximately twice the design flow), rock was moved from the crest and scattered onto the downstream apron so that the resulting configuration was a rock sill with a triangular cross section having a 1 on 1 upstream slope and a 1 on 10 downstream slope. Two important observations were made during these latter tests. First, the failure occurred at the crest and not at the sill side slopes as had been previously feared, and second, even though the rock was rearranged, actual destruction of the rock sills never occurred and they still created considerable head loss.

The rock sills, fencing, and earthen groins were constructed as planned and willows were also planted in the berm areas to induce siltation. All structures placed during initial construction phases have functioned as planned. Following the initial construction, which was completed in 1969, downstream degradation required the construction of four additional rock sills and fencing in 1973 at the lower end of the project. The channel below these structures has continued to degrade slowly. Degradation below rock sill 1D is between 4 and 5 feet and has undercut the fencing. This may eventually cause failure of the fencing and erosion of the channel banks.

ITEM 18-1
(Sheet 6 of 6)

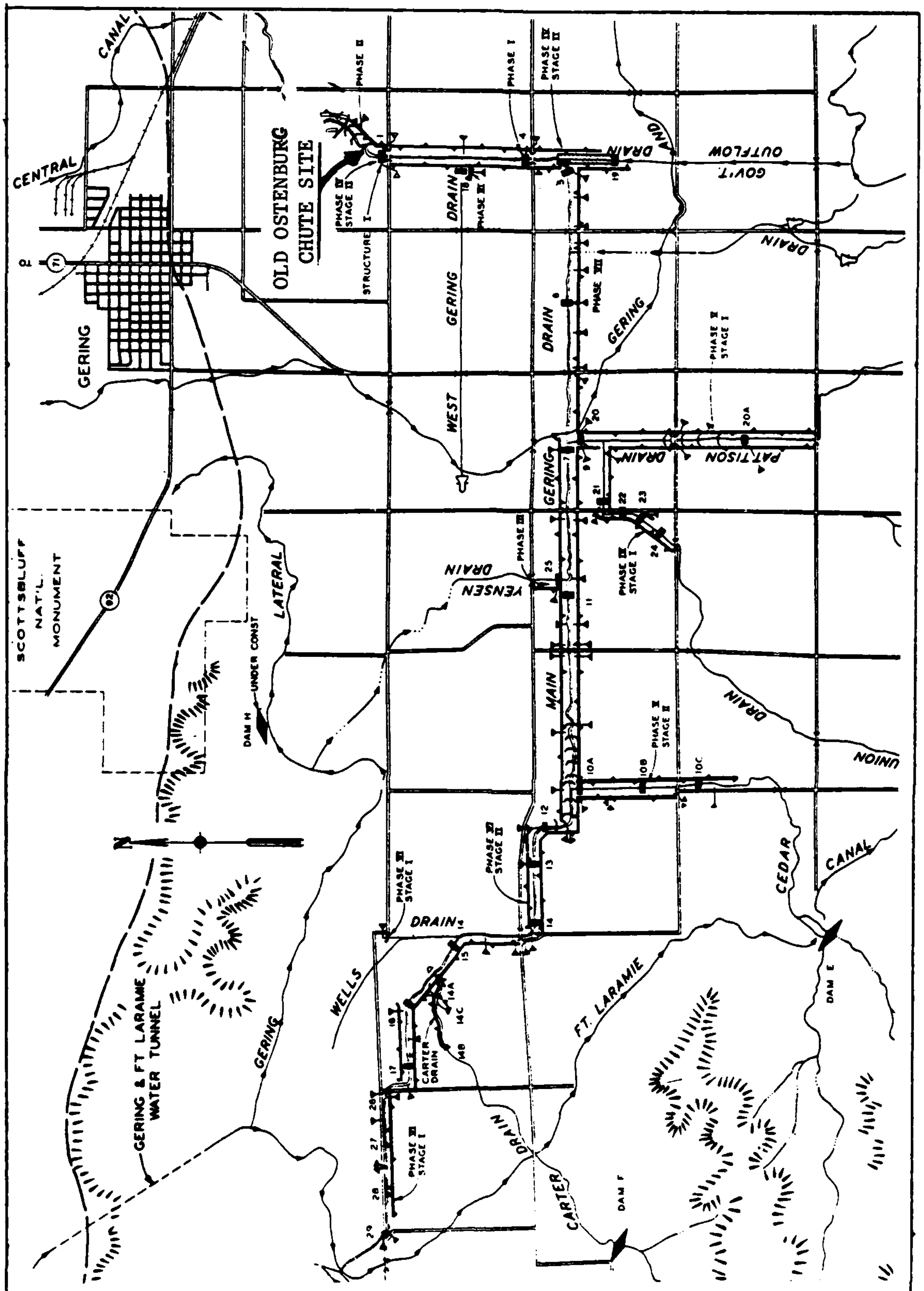

Figure 1. Gering Drain near Gering, Nebraska

ITEM 18-2

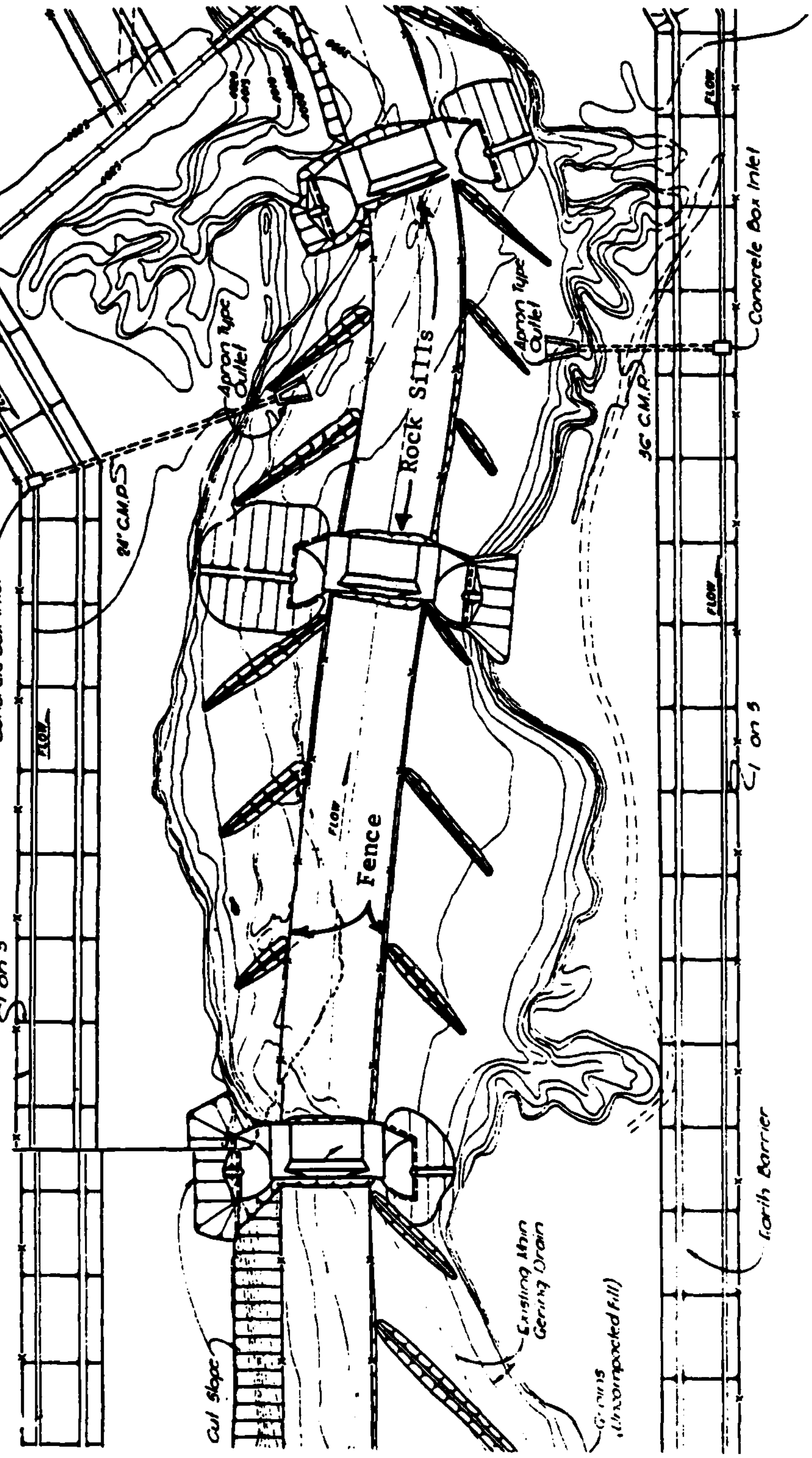

Figure 2. Gering Drain near Gering, Nebraska. Typical layout of low rock sills and fencing.

ITEM 18-3

Figure 3. Gering Drain near Gering, Nebraska.
View upstream of a low rock sill.

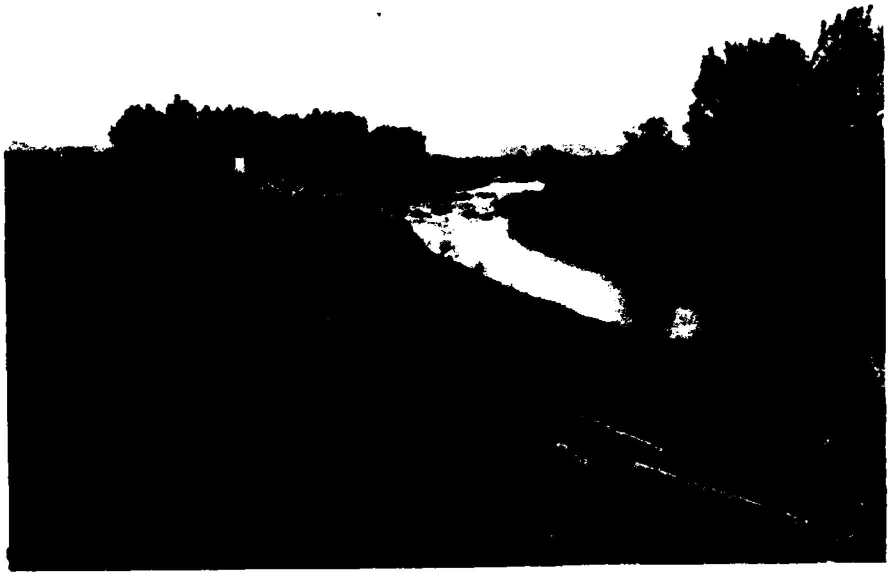

Figure 4. Gering Drain near Gering, Nebraska.
View of low rock sill and fence line with
stabilized channel.

ITEM 18-4

LITTLE SIOUX RIVER
ONAWA, IOWA

Streambank Erosion Control Evaluation and Demonstration Act of 1974
Section 32 Program - Work Unit 2

EVALUATION OF EXISTING BANK PROTECTION WORKS

(1) Location

Stream Little Sioux River River Mile 5.7 Side ________

Local Vicinity ________ Lat N42°0' Long W96°10'

At/Nr City Onawa County Monona State IA Cong Dist 6

CE Office Symbol MRO Responsible Agency Corps of Engineers

Site Map Sources Omaha District, Corps of Engineers

Land Use Homes and farming.

(2) Hydrology at or Near Site

Stage Range ________ to ________ ft; Period of Record 1958 to 1981.

Discharge Range 22 to 30,000 cfs; Velocity Range ________ to ________ fps

Sediment Range 12 to 1,120,000 tpd; Period of Record 1959 to 1969.

Bank-full Stage ________ ft; Flow 39,600 cfs; Average Recurrence Interval 50 yr

Bank-full Flow Velocity: Average ________ fps; Near Bank ________ fps

Comments Bed gradient 1 ft/mile.

(3) Geology and Soil Properties

Bank (USCS) Fat and lean clays Bed (USCS) Fat and lean clays

Data Sources Corps of Engineers test borings.

Groundwater Bank Seepage ________

Overbank Drainage Discharges above 9,000 cfs overtop grade-control sturcture

Comments ________

(4) Construction of Protection

Need for Protection Flows above 9000 cfs bypass grade-control structures and reenter degrading channel downstream as overbank drainage.

Erosion Causative Agents See above.

Protection Techniques Gabion mattress.

General Design Grouted riprap on slopes with 150 by 50 ft gabion mattress side slopes downstream of stilling basin.

Project Length ________ ft; Construction Cost $ 25,000* Mo/Yr Completed 1969

*For gabion mattress only.

(5) Maintenance

Experienced Flows (Stage, cfs, Date) Maximum 30,000 cfs 19 Feb.1971

Repairs and Costs (Item, Cost, Data) Repair and grout baskets, $22,000, 1973; repair and grout baskets, $6,000, 1975.

Comments:

(6) Performance Observations and Summary

Monitoring Program Visual inspection.

Documentation Sources Corps of Engineers, Omaha District

Project Effect on Stream Regime

Project Effect on Environment

Successful Aspects

Unsuccessful Aspects Right bank mattress damaged; left bank mattress completely failed in 1979.

General Evaluation Entire structure is currently under study for possible design change.

Recommendations

(7) Additional Information, Comments, and Summary

Map No. 19.

Attached Items:

19-1 Project summary
19-2 Project location
19-3 Plan view
19-4-6 Photographs
19-7 Temporary protection measures

Little Sioux River at Onawa, Iowa (Mile 5.7)

The Little Sioux River has headwaters in Jackson County, Minn., and flows in a southwesterly direction until the stream discharges into the Missouri River main stem near mile 669.2. The Monana-Harrison Drainage District dug an equalizer ditch between the Little Sioux and the Monona-Harrison Ditch in the early 1900's, Figure 1. As a result the Monona-Harrison Ditch captured the discharge of the Little Sioux such that little or no flow was passed below the diversion. As part of the Little Sioux River Basin Flood Control Project, the equalizer ditch was closed and the Little Sioux was rebuilt downstream from the diversion. During flood periods, discharge can now be exchanged between the Little Sioux and Monona-Harrison Ditch via an upstream diversion channel.

The USGS has operated a gaging station on the Little Sioux near Turin, Iowa (mile 13.5), since January 1958, Figure 1. The daily discharges of record (1958 to the present) are: maximum 30,000 c.f.s. (19 February 1979), mean 1,085 c.f.s., and minimum 22 c.f.s. (10-22 February 1959). The Corps of Engineers operated a daily suspended-sediment sample collection station at the same location from March 1959 through July 1969. Daily suspended-sediment loads of record were: maximum 1,120,000 tons (1 June 1959), mean 10,414 tons, and minimum 12 tons (on several days). The maximum annual suspended-sediment load of record was 6,694,200 tons (water year 1962); the average annual load was 3,801,130 tons. Average annual sediment yields over the lower Little Sioux Basin are 6,000 to 10,000 tons/square mile.

Prior to the placement of three grade-control structures at the mouth of the Little Sioux in 1959, Figure 1,* channel degradation had progressed approximately 3.5 miles upstream from the mouth. Between

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* Part of the construction under the Little Sioux River Basin Flood Control Project.

ITEM 19-1
(Sheet 1 of 3)

1959 and 1964 the headcutting advanced another 2.5 miles or a total of 6 miles. The degradation had thus progressed to the point where it was no longer practical to control the advance by increasing the stage in the lower reach of the stream through the use of additional grade-control structures at the mouth. The proposed method of halting the headcutting was to place a control structure just downstream of the upper limits of the serious erosion. The grade-control structure (designated #4 in Figure 1) consisted of a 50-ft-wide rectangular concrete drop structure in the central channel flanked by upstream rock sills on 158-ft-wide berms extending to the levees, Figure 2. The structure was built using conventional riprap following recommendations from a Waterways Experiment Station model study. The structure (completed in 1964) was designed to withstand a discharge of 35,000 c.f.s. at stream velocities up to 18 fps; with the structure in place the bed gradient through this reach was adjusted to 1 ft/mile.

The structure was designed such that when flows greater than 8,000 c.f.s. occurred, the upstream channel berms were overtopped, causing overbank flows downstream of the drop. This flow reentered the downstream channel over the riprap on the side slope downstream of the stilling basin. Additional downstream degradation caused more lowering of the tailwater than was anticipated and resulted in severe problems with the reentrant overbank flows. Damage to the riprap due to high flows in April 1965 (27,100 c.f.s. maximum daily flow) required placement of grouted derrick stone (800-1,000 lb) adjacent to and downstream of the stilling basin to repair the structure, Figure 3. Further flows undermined sections of the grouted stone revetment. In 1969, a 150- by 50-ft gabion mattress was placed on the left and right banks of the stilling basin downstream of drop structure, Figure 2. Each mattress consisted of interwoven rock-filled baskets with 12 x 3 x 1-ft dimensions. The outer edge of each mattress consisted of a single row of 6 x 3 x 3 ft baskets to tie down the perimeter of the mattress. The gabions were specified to have physical properties equivalent to those gabions

ITEM 19-1
(Sheet 2 of 3)

sold by Terra Aqua, Inc., of Reno, Nev.; no other specifications are available except that the gabion wire was to be galvanized. The stone placed in the baskets was required to have a gradation of 4 to 8 in. The total cost for placement of the two mattresses was $25,000 (1969). The mattresses were damaged in 1973 by return flows and some of the baskets had to be replaced; in addition, a 2- to 3-in. layer of grout was placed over the mattress surfaces to prevent debris from hanging up in the mesh and subsequently breaking the wire Also the grout prevented rust. The wire was galvanized; however, fishermen often built fires on the exposed mattress which removed the protective coating, thus making the metal susceptible to oxidation. The total cost for the 1973 repairs was $22,000. Again, in 1975, several baskets had to be replaced and grouted ($6,000). The two gabion mattresses were selected as a Section 32 existing site.

During an inspection visit in September 1978, failures were observed on the right bank grouted stone section, Figure 4, and on the left bank mattress. The mattress damage included loss of grout, Figure 5, and failure of part of the mattress under its own weight due to scour under the mattress, Figure 6. Snowmelt runoff in March 1979 resulted in overbank return flows that partially failed the right bank mattress, Figure 7, and completely failed the left bank mattress, Figure 8. These return flows also caused large scour areas to develop on both banks, Figure 9, which threaten to flank the structure. Flows during this period were mostly between 10,000 to 20,000 c.f.s., with a peak flow of 24,000 c.f.s. There is no plan to repair either mattress as the entire structure may have to be redesigned. In April 1980, as a precaution, diversion dikes were built upstream of the structure to prevent all but extreme flows from bypassing the control structure. Although the structure has successfully stopped further upstream headcutting, the width of the weir is apparently not sufficient. One proposal currently being considered is placement of two additional weirs on either side of the existing structure at slightly higher elevations.

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(Sheet 3 of 3)

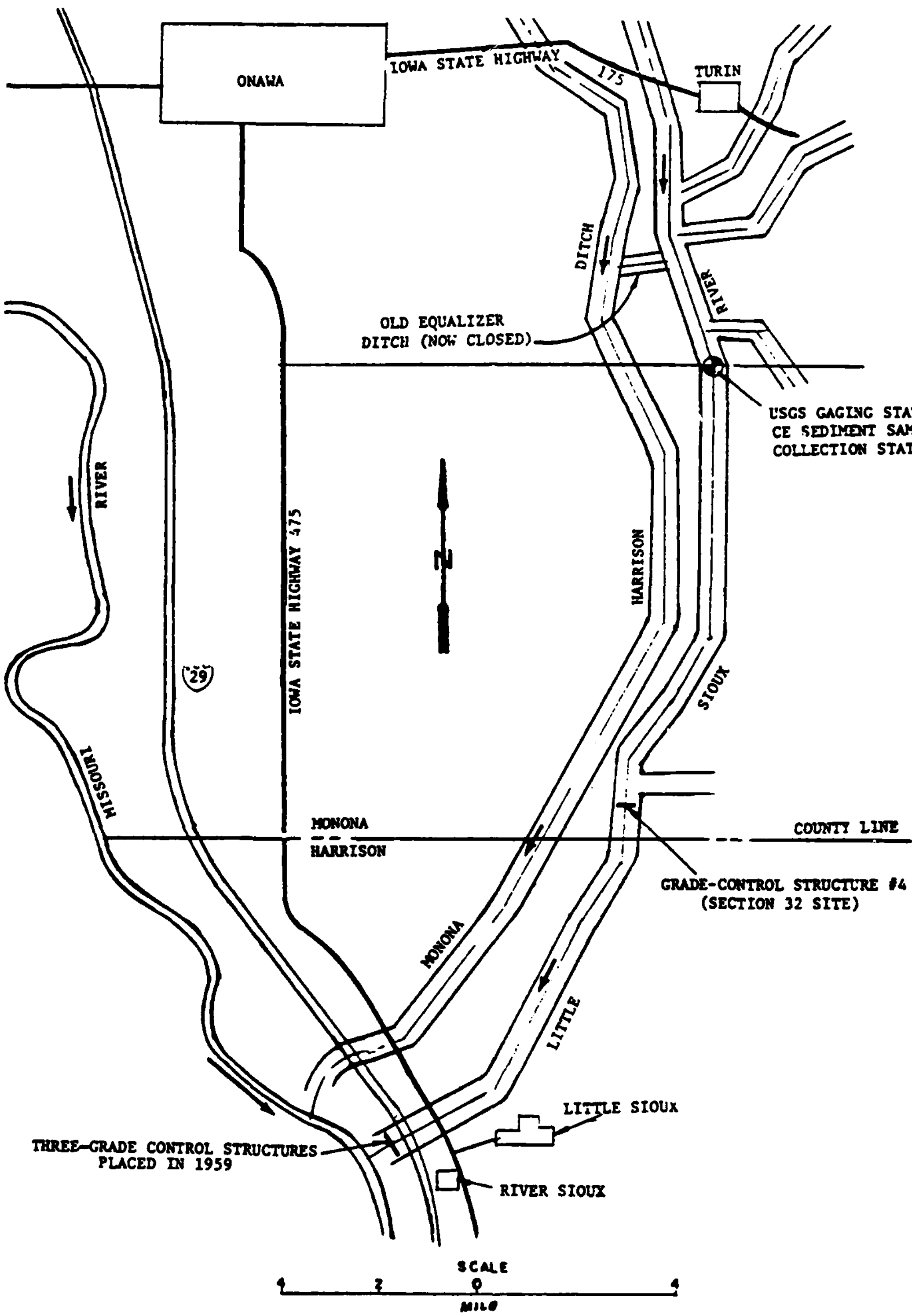

Figure 1. Little Sioux River at Onawa, Iowa. Heavy lines paralleling streams represent levees

ITEM 19-2

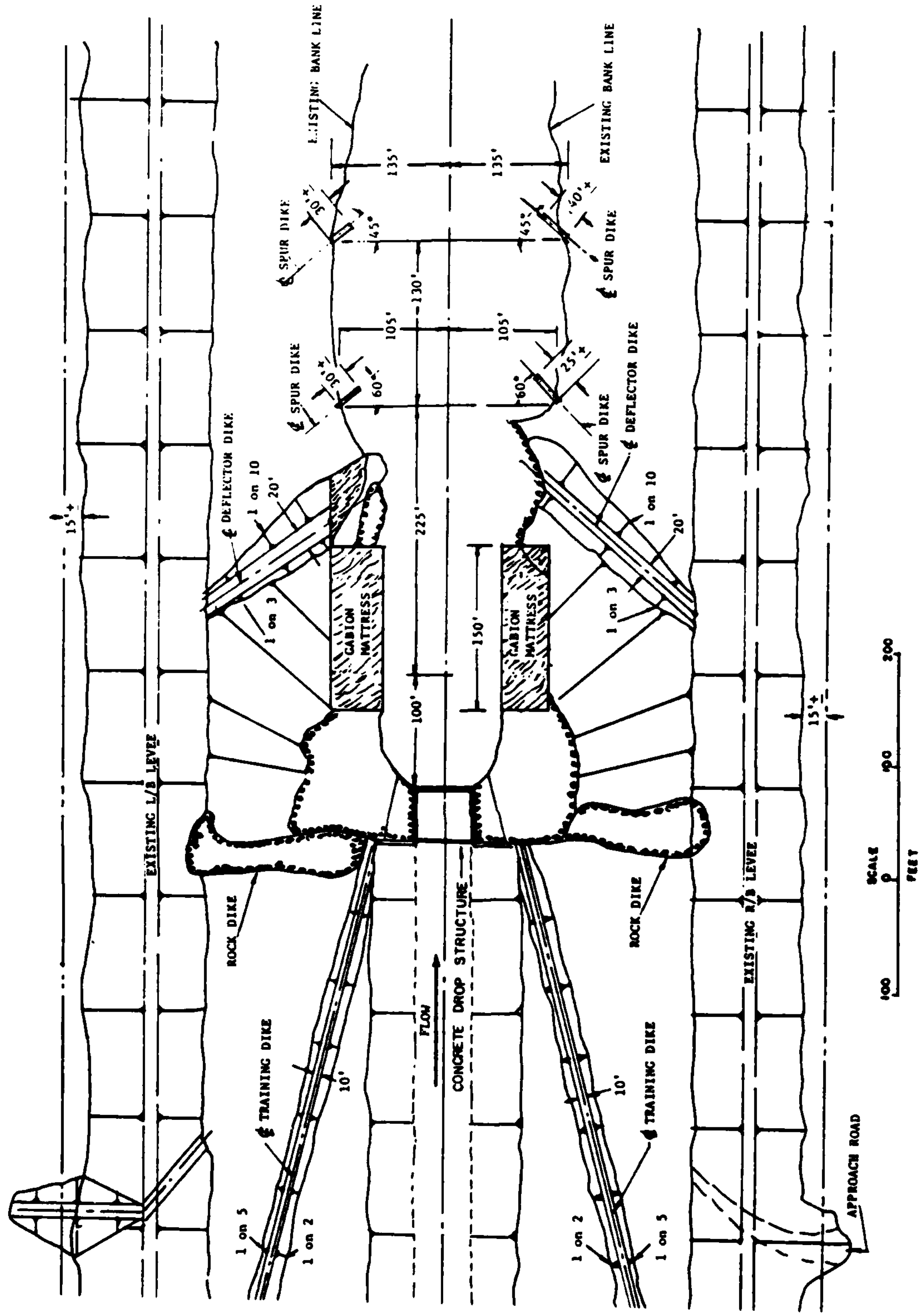

Figure 2. Little Sioux River at Onawa, Iowa. Plan view of grade-control structure #4 and gabion mattresses

ITEM 19-3

Figure 3. Little Sioux River at Onawa, Iowa. Damage to the stilling basin riprap as a result of high flows in April 1965 required the placement of grouted derrick stone to repair the basin

Figure 4. Little Sioux River at Onawa, Iowa. Toe failure of right-bank grouted stone section (20 September 1978)

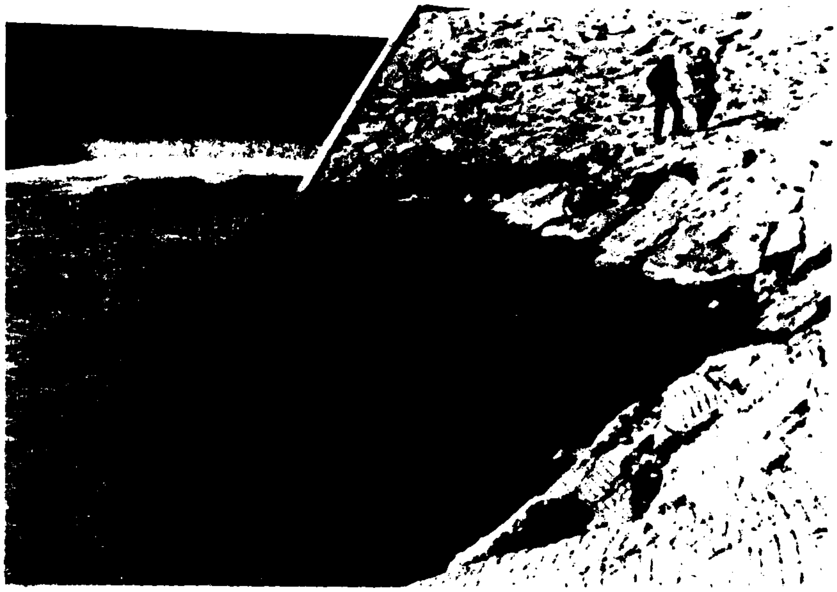

Figure 5. Little Sioux River at Onawa, Iowa. Loss of grout on left-bank mattress

Figure 6. Little Sioux River at Onawa, Iowa. Part of the left-bank mattress failed under its own weight, due to scour under the mattress

Figure 7. Little Sioux River at Onawa, Iowa. Overbank return flows during March 1979 partially failed the right-bank mattress (July 1979)

Figure 8. Little Sioux River at Onawa, Iowa. Overbank return flows during March 1979 completely failed the left-bank mattress (July 1979)

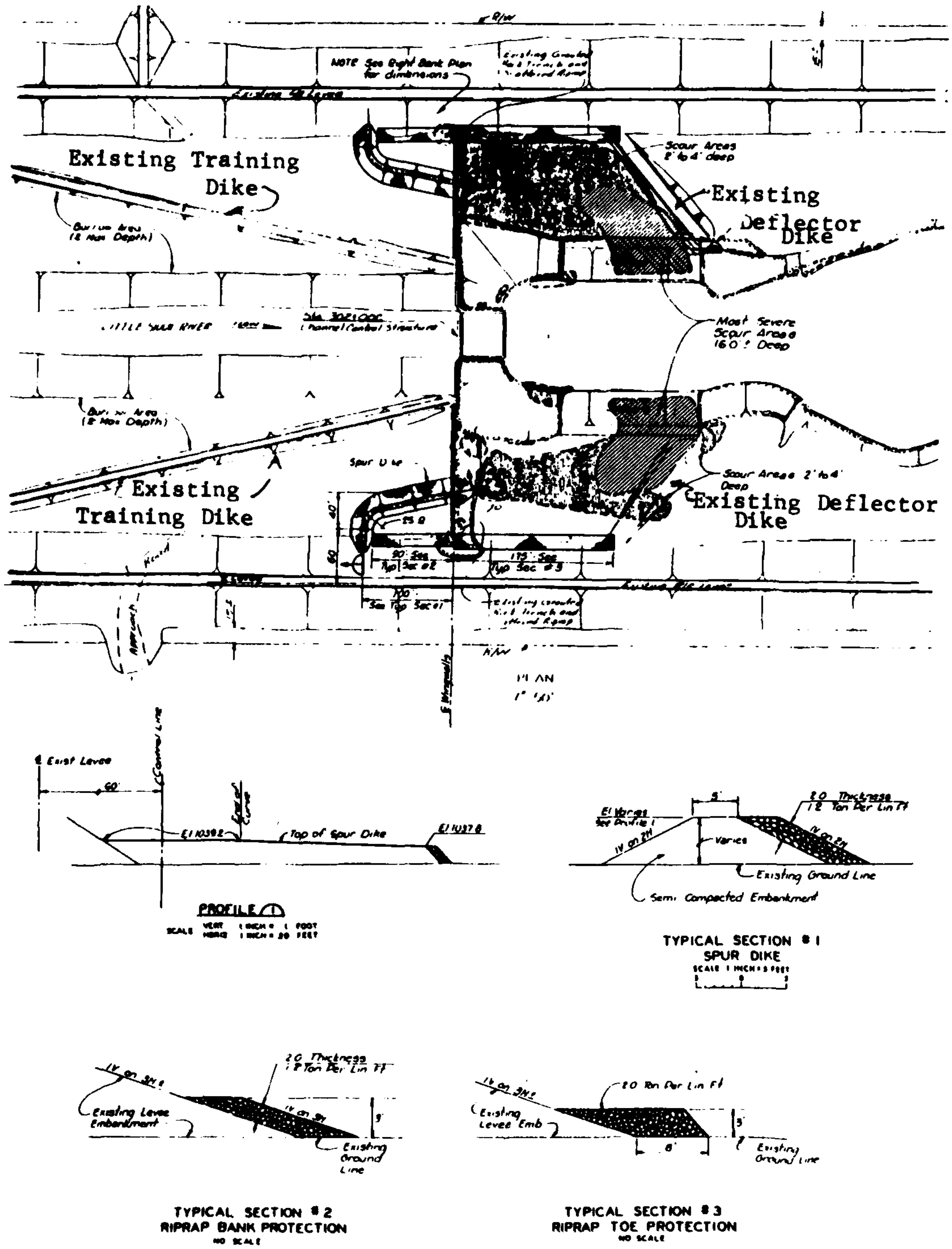

Figure 9. Little Sioux River at Onawa, Iowa. Temporary protection measures (spur dikes) placed in April 1980.

ITEM 19-7

DEADMAN'S RUN AND ANTELOPE CREEK
LINCOLN, NEBRASKA

Streambank Erosion Control Evaluation and Demonstration Act of 1974
Section 32 Program - Work Unit 2

EVALUATION OF EXISTING BANK PROTECTION WORKS

(1) Location

Stream Deadmans' Run & Antelope Creek River Mile ______ Side ______

Local Vicinity Southeast Nebraska Lat N49°45' Long W 96°36'

At/Nr City Lincoln County Lancaster State NE Cong Dist 1

CE Office Symbol MRO Responsible Agency Lower Platte South Natural Resource District

Site Map Sources ______

Land Use Information Highly urbanized.

(2) Hydrology at or Near Site

Stage Range NA to ______ ft; Period of Record 1958 to 1962.

*/Discharge Range 0 to 2,800 cfs; Velocity Range NA to ______ fps

Sediment Range NA to ______ tpd; Period of Record 19__ to 19__.

Bank-full Stage NA ft; Flow ______ cfs; Average Recurrence Interval ______ yr

Bank-full Flow Velocity: Average ______ fps; Near Bank ______ fps

Comments Bed gradient 30-14 ft/mile

(3) Geology and Soil Properties

Bank (USCS) Fat clay, to lean sandy clay Bed (USCS) CH, CL, & SC

Data Sources ______

Groundwater Bank Seepage Isolated areas and storm drainage culvert intersections

Overbank Drainage ______

Comments ______

(4) Construction of Protection

Need for Protection To protect urban development.

Erosion Causative Agents Channel straightening with resultant velocity increases and degradation.

Protection Techniques Gabions

General Design One course of gabions placed on compacted streambed and against shaped bank.

Project Length 26,700 ft; Construction Cost $ ______ Mo/Yr Completed Still unde construct:

(5) Maintenance

Experienced Flows (Stage, cfs, Date) ______

Repairs and Costs (Item, Cost, Data) Limited to vegetation removal and repair of vandalism; costs not available.

Comments: ______

(6) Performance Observations and Summary

Monitoring Program Visual inspections.

Documentation Sources ______

Project Effect on Stream Regime Stabilized banks.

Project Effect on Environment Nothing adverse.

Successful Aspects Has stabilized channels and banks.

Unsuccessful Aspects Gabion baskets subject to vandalism damage through highly urbanized area.

General Evaluation Project has been subject to 20-yr. flood with no apparent damage.

Recommendations */Estimated maximum discharge on Deadman's Run 1,500 cfs Figures shown are for Antelope Creek.

(7) Additional Information, Comments, and Summary

Map No. 20. Channels in one area was previously stabilized with concrete slab rectangular channel liner which failed when weep holes became plugged and excessive uplift pressures developed.

Attached Items:

20-1 Project summary and location
20-2 Project cost and construction dates 20-4-5 Photographs
20-3 Gabion details

Deadman's Run and Antelope Creek at Lincoln, Nebr.

Deadman's Run and Antelope Creek are right-bank tributaries of Salt Creek (Salt Creek miles 26.0 and 28.0, respectively) at Lincoln, Nebr. Salt Creek is a right-bank tributary of the Platte River, entering that stream near Ashland, Nebr. Deadman's Run rises east of the Lincoln city limits and flows northwesterly 7 miles where it enters Salt Creek just downstream from the Chicago and Northwestern Railway bridge in north-central Lincoln, Figure 1. The 10.5-square-mile watershed is located entirely within Lancaster County, Nebr., having a length of about 6 miles and a maximum width of 2.5 miles. Topographically, the watershed is divided at the Holdredge Street Bridge. Downstream from this crossing, the streambed gradient is approximately 14 ft/mile; however, upstream the gradient ranges from 30 to 40 ft/mile. Watershed elevations in the Deadman's Creek project area range from 1,117 to 1,195 ft.

The headwaters of Antelope Creek are located near Cheney, Nebr. From Cheney, Antelope Creek flows in a northwesterly direction 10 miles to join Salt Creek at the State Fair Grounds in Lincoln, Figure 1. The basin length is 8 miles with an average width of 2.5 miles. The total area of the drainage basin (entirely within Lancaster County, Nebr.) is 14 square miles, of which 5.4 square miles is controlled by Antelope Creek Dam, constructed by the Corps of Engineers in 1962. The basin's topography ranges from moderately rolling hills in the lower reaches to steeply rolling hills in the upper reaches. The streambed gradient averages 16 ft/mile from the confluence with Salt Creek to Antelope Creek Dam; above this impoundment the gradient increases to 28 ft/mile. Watershed elevations in the Antelope Creek project reach range from approximately 1,120 to 1,215 ft.

There is no long-term gaging information available for Deadman's Run. Records list the floods of 2 and 14 June 1951, and the flood of 25 June 1963, as being major events. During the flood of 2 June 1951,

an area equivalent to approximately 40 city blocks was inundated with approximately 50 residences being underwater to some extent; no discharge or damage estimates were made. The 14 June 1951 storm flooded 43 city blocks between the northern edge of the University of Nebraska and the Chicago, Rock Island and Burlington Northern Railroad tracks, Figure 1; the peak discharge was estimated to be 1,500 c.f.s. Floodwaters remained in the basin for about 5 hours and inundated 52 homes. Damages resulting from this flood event were estimated (1951) to be $47,000. The 25 June 1963 storm in Deadman's Run was referred to as being "intense"; although no discharge estimates were made, near bank-full flow was observed. Damages were confined to the Cornhusker Highway Bridge, Figure 1, and were estimated (1963) to be $27,000.

The USGS operated a stream-gaging station on Antelope Creek at Lincoln (mile 0.7) from 28 June 1958 through 30 September 1962. Daily discharges of record were: maximum 2,800 c.f.s. (10 July 1958), mean 4.46 c.f.s., and minimum no flow (7 and 21 December 1958). The 1958 flood of record inundated 135 homes and a number of business establishments; however, no damage estimates are available. Prior to the period of record, there were a number of floods that occurred in the Antelope Creek Basin including those of 1908, 1910, 1940, 1950, 1951, 1952, and 1957. The two most significant of these were 14 June 1951 and 27 June 1952. During the 1951 event, heavy rains over the basin caused inundation of about 760 acres. The flood conditions were aggravated by bridge constrictions and floating debris. Damages to 92 commercial buildings, 298 homes, streets, and railroads were estimated to be $472,000 (1951). The storm of 1952 was less severe, damaging only five commercial structures and ten residences.

No suspended-sediment load data are available for either stream; however, average annual sediment yields over the basins range from 1,000 to 3,000 tons/square mile. Surface soils along the streams are generally fat clays (CH) and lean and sandy clays (CL).

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Both Deadman's Run and Antelope Creek are naturally meandering streams; however, these two creeks are not in their original channels due to residential and commercial encroachments. During the development of adjacent areas, the channels were straightened into new alinements which resulted in higher channel velocities. The increased velocities tended to develop wider streambeds and to encourage channel degradation. This erosion created 20-ft-high near-vertical banks at many locations, which in turn caused much concern among local residents not only because of the loss of property, but because of the safety hazard. To mitigate these problems, the Lower Platte South Natural Resources District (LPSNRD) developed a program to control the erosion and water flow. Initially, a concrete slab rectangular channel liner was placed in a small test reach; however, weep hole plugging caused excessive uplift pressure, resulting in damage to the structure. Because the concrete liner proved to be unstable, gabions were selected to revet the banks of both streams not only to provide adequate bank drainage, but to allow placement of nearly vertical structures that would minimize top-bank property losses.

Initial bank stabilization to sustain a 100-year flood event began in 1969 and continues to the present. Through 1979, 11 construction reaches have been completed in Deadman's Run, having a total project length of 15,201 ft, Figure 1 and Table 1. Six reaches have been completed in Antelope Creek and one is now under construction; these seven reaches will total 11,537 linear ft in length. The projects in both streams were designed for LPSNRD by Clark Enerson Partners of Lincoln, with the onsite construction being completed by various local contractors. The 18 construction reaches have been collectively selected as a Section 32 existing site.

Prefabricated gabion cages have been marketed in Europe for many years; however, gabions for the construction of bank protection works in the United States have been used widely only in the past 20 years. The

basic element of the gabion revetment is the cage or "basket." The cage is a wire mesh structure divided by diaphragms into cells, Figure 2. The gabions used for this project were obtained from Maccaferi Gabions, Inc. The mesh was fabricated from U.S. Steel Wire Gauge #11 zinc-coated, galvanized wire. The tensile strength of the wire was specified to be in the range of 60,000 to 85,000 psi, with a minimum zinc coating on the wire of 0.80 oz/sq ft. The maximum dimension of the mesh opening could not exceed 4-1/2 in. and the area of the mesh opening not larger than 8 sq in. The wire mesh was required to have sufficient elasticity to permit elongation of the mesh equivalent to a minimum of 10 percent of the length of the section under test without reducing the gage or tensile strength of the individual wires to a value less than that for similar wire, one gage smaller in diameter. The gabions were shipped flat and wired together onsite, Figure 2. After assembly, the cages were filled with stone. The stone was specified to have not more than 5 percent by weight of the total material passing a 3-in. sieve and not more than 10 percent by weight of the material retained by a 12-in. sieve. The maximum weight for any one stone could not exceed 40 lb and have no dimension less than 3 in. or greater than 16 in. In addition, the stone was specified to be of a composition suitable to withstand abrasion, to be nonfriable, and to be resistant to weathering and freeze-thaw actions.

Prior to gabion placement, the bank was shaped and the streambed compacted. A gabion support apron was then placed on the compacted material, Figure 3. The aprons served to protect the toe of the revetment and to distribute some of the load of the gabion baskets which would be stacked vertically on the apron. The aprons have dimensions of 1 ft vertical, 3 ft wide (parallel to bank), and 6, 9, or 12 ft long, depending on how much of channel bed was to be covered. At locations where scour was possible, such as culvert outlets, bridge piers, etc., the entire bottom of the channel was lined.

After completion of the support apron, the first course of gabions was placed. The baskets were filled with stone by machine, and then hand-arranged to minimize voids. The stone on the front face of the gabion, Figure 3, was stacked to present a pleasing appearance, Figure 4. Noncorrosive wire was used to fasten the gabions together and to the support apron to further strengthen the structure. A minimum of a 2-ft width of fill material was placed behind the gabions and compacted. To minimize overall project costs, only one course of gabions was used where possible. The top face of the gabion course was then covered with fill material which was then sloped back at 1V on 3H toward intersection with the natural terrain profile. For reaches where the surface area along the top bank was at a premium (residences, commercial concerns, etc.), gabion courses were stair-stepped as high as practical to maximize the available top-bank surface area, Table 1 and Figure 5. Filter material such as sand, gravel, or fabric was generally not used below the apron or between the gabions and the bank. The exceptions were known moist banks where leaching of the soil through the gabions was possible and at intersections of the stream with drainage culverts where fabric was placed under the support apron in the area of the culvert discharge impact. Celanese Mirafi 140 or Dupont Typar were recommended for use by Clark Enerson Partners, although local contractors were at liberty to make their own selection.

The upper bank was seeded with reed canary grass (5.0 lb/acre) in all areas except parks, where a bluegrass mixture (50 lb/acre) was used. Although reed canary grass has a superior root system, the bluegrass has a much more pleasing appearance and is more easily maintained. After seeding, the exposed area was mulched with clean native hay at the rate of 2-1/2 tons/acre.

At the time of the inspection by the Waterways Experiment Station team, the projects were performing well. Maintenance has been limited to vegetation clearance, Figure 6, and replacement of stone removed by

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vandals; maintenance costs are not available. The project has not been tested by the 100-year design flood (bank-full condition), but was subjected to a 20-year flood in September 1977 with no apparent damages. No discharge estimates are available for this event; however, bank-full conditions were not approached at any location. At locations where gabions extended only a part of the way to the top of the bank, gabions were overtopped through several reaches.

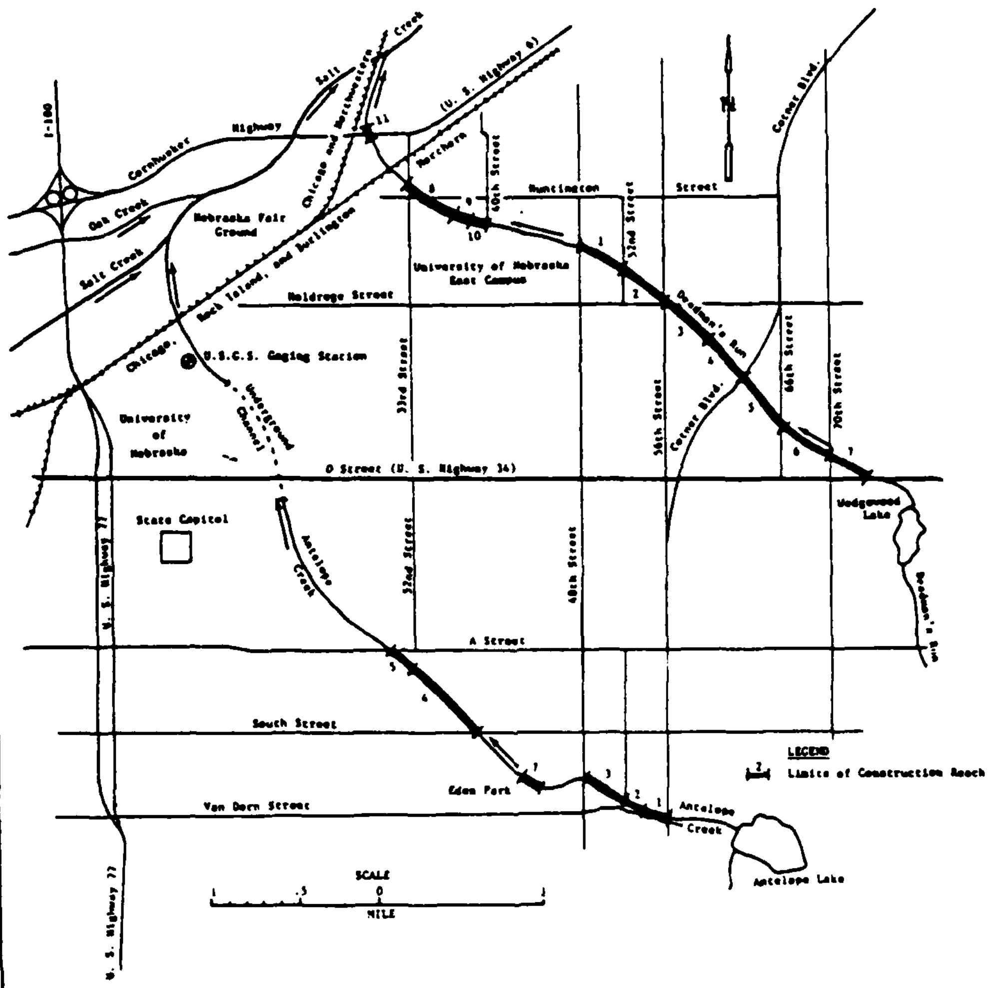

Figure 1. Deadman's Run and Antelope Creek at Linclon, Nebr.

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Summary of Gabion Placement in Deadman's Run and Antelope Creek

Construction Reach	Construction Schedule Started	Construction Schedule Completed	Completed Cost* ($)	Project Length, ft	No. of Gabion Courses**
		Deadman's Run			
1 - 48th to 52nd Streets	1970	1970	166,831	1,750	5+
2 - 52nd to 56th Streets	1970	1970	196,760	1,750	5+
3 - 56th to 60th Streets	1973	1974	218,747	1,950	1
4 - 60th Street to Cotner Blvd	1969	1970	85,346	1,750	1
5 - Gateway	1971	1972	86,245	1,800	1
6 - 66th to 70th Streets	1971	1972	139,463	1,800	2+
7 - 70th to "O" Streets	1971	1972	83,106	950	2
8 - 33rd & Huntington Streets	1973	1975	189,732	1,831	1+
9 - East Campus Bridge	1973	1974	89,850	800	2
10 - East Campus Bridge to 40th Street	1978	1979	63,311	500	1+
11 - Cornhusker Highway	1971	1972	22,452	205	2
				15,201	
		Antelope Creek			
1 - 52nd to 56th Streets	1971	1971	30,741	930	1
2 - 52nd to Van Dorn Streets	1976	1977	53,795	535	1
3 - 48th to 52nd Streets	1977	1978	179,447	1,967	1
4 - South to 33rd Streets	1971	1973	268,849	3,600	2+
5 - 33rd to "A" Streets	1969	1972	148,623	1,955	2
6 - Nebraska State Fairgrounds	1973	1974	223,933	1,750	2
7 - Eden Park	1979	Inc.	79,750	800	-
				11,537	

* Includes cost for clearing, grubbing, excavation, backfill, grading, gabions and rock-fill material, seeding, and some sewer construction.

** A plus means that more gabion courses than the number indicated were used at some locations.

Table 1

ITEM 20-2

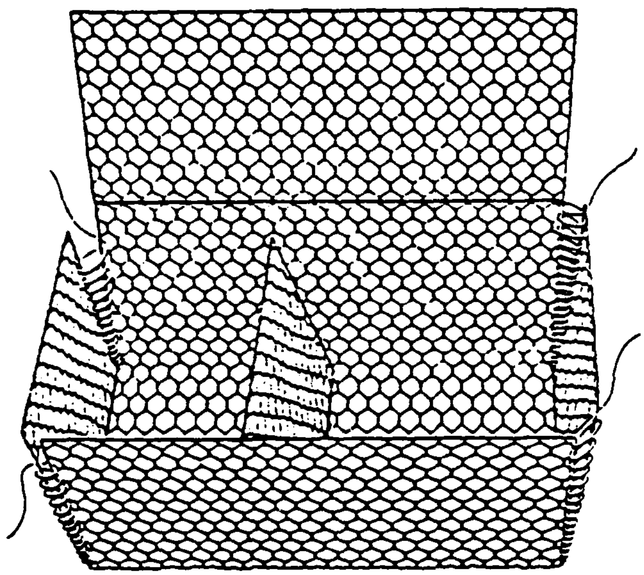

Figure 2. Deadman's Run and Antelope Creek, Lincoln, Nebraska. Typical gabion cage used for revetment construction.

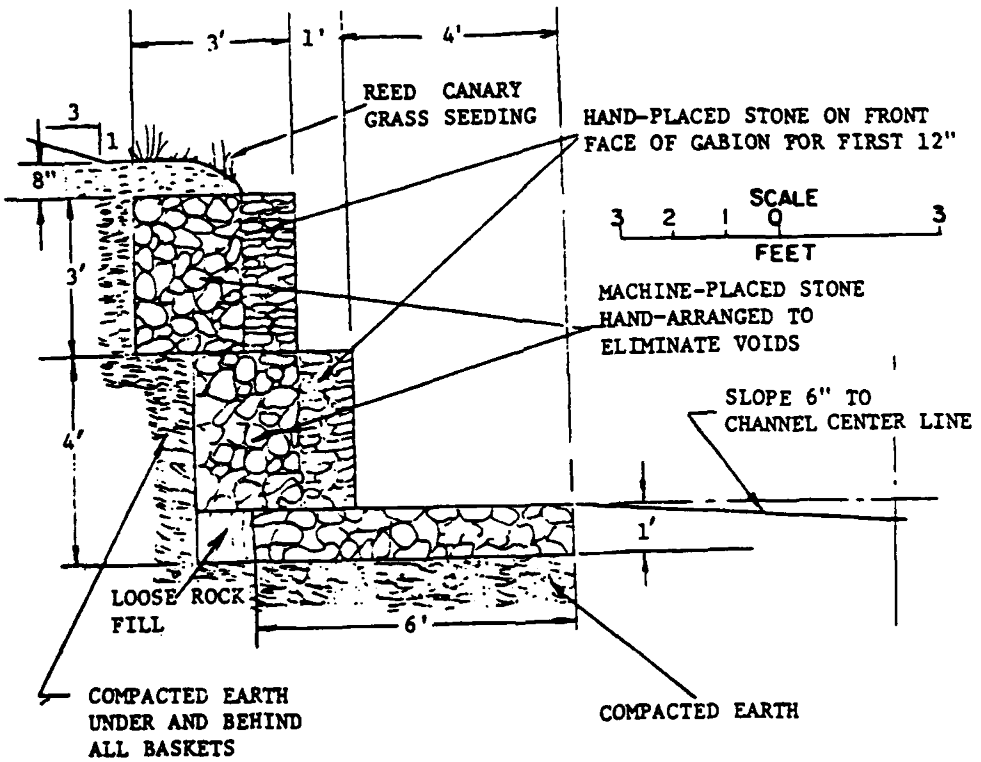

Figure 3. Deadman's Run and Antelope Creek, Lincoln, Nebraska. Typical gabion revetment.

Figure 4. Deadman's Run and Antelope Creek, Lincoln, Nebraska. The stone on the front face of the gabions was stacked to present a pleasing appearance.

Figure 5. Deadman's Run and Antelope Creek, Lincoln, Nebraska. Gabion courses were stacked vertically in a stair-step manner at several locations to provide maximum top-bank surface area.

Figure 6. Deadman's Run and Antelope Creek, Lincoln, Nebraska. Removal of vegetation is sometimes required to maintain channel efficiency

FLOYD RIVER
SIOUX CITY, IOWA

Streambank Erosion Control Evaluation and Demonstration Act of 1974
Section 32 Program - Work Unit 2

EVALUATION OF EXISTING BANK PROTECTION WORKS

(1) Location

Stream Floyd River River Mile 0.1-1.6 Side ______

Local Vicinity ______ Lat N42°30' Long W96°20'

At/Nr City Sioux City County Woodbury State IA Cong Dist 6

CE Office Symbol MRO Responsible Agency Corps of Engineers

Site Map Sources Omaha District, Corps of Engieers

Land Use Information Highly urbanized

(2) Hydrology at or Near Site

Stage Range ______ to ______ ft; Period of Record 1934 to 1981.

Discharge Range 0.9 to 71,500 cfs; Velocity Range 1 to 14 fps

Sediment Range 0 to 171,000 tpd; Period of Record 1968 (54) to 1973 (57).

Bank-full Stage ______ ft; Flow 71,500 cfs; Average Recurrence Interval 100 yr

Bank-full Flow Velocity: Average 14 fps; Near Bank ______ fps

Comments Project designed for 71,500 cfs at 12 fps. Bed gradient 4 ft/mile approximately 2 feet of freeboard.

(3) Geology and Soil Properties

Bank (USCS) Lean and fat clays Bed (USCS) Lean and fat clays 1/

Data Sources Corps of Engineers test borings.

Groundwater Bank Seepage Project area has underlying layer of impervious material

Overbank Drainage Controlled

Comments 1/ Highly erodible sands and gravels a few feet below bed.

(4) Construction of Protection

Need for Protection To prevent channel degradation.

Erosion Causative Agents Channel straightened which would cause degradation of the bed.

Protection Techniques Five grade-control structures 2,000 feet apart.

General Design Steel-sheet piling curtain with large stone paving upstream and downstream.

Project Length 8,000 ft; Construction Cost $ * Mo/Yr Completed 7/66

*Costs of grade structures not separable from total project costs.

(5) Maintenance

Experienced Flows (Stage, cfs, Date) 17,000 cfs April 1969

Repairs and Costs (Item, Cost, Data) None to date.

Comments:

(6) Performance Observations and Summary

Monitoring Program Visual inspections, resurvey.

Documentation Sources Corps of Engineers, Omaha District

Project Effect on Stream Regime Has prevented channel degradation.

Project Effect on Environment

Successful Aspects

Unsuccessful Aspects Sheet pile in structure at Sta. 2+90 has been bowed downstream in center.

General Evaluation Project overall is functioning as designed.

Recommendations

(7) Additional Information, Comments, and Summary

Map No. 21.

Attached Items:

21-1 Project summary
21-2 Project location
21-3 Typical plan
21-4 Half sections
21-(5-6) Photograph and half section

Floyd River at Sioux City, Iowa (Mile 0.1 to 1.6)

The Floyd River, located in northwestern Iowa, is a left-bank tributary of the Missouri River at Sioux City, Iowa (Missouri River mile 731.2). The Floyd River Basin is comparatively long and narrow with a total drainage area of 956 square miles. For much of its course, the river flows in a broad alluvial valley, with bottomlands being 3,000 to 3,500 ft wide in the vicinity of Sioux City. Although the valley walls are gently sloping farther upstream, the valley is flanked by high, steep-sided loessial bluffs as the stream approaches its confluence with the Missouri River.

No discharge or sediment data have been collected on the Floyd River at Sioux City; however, the USGS has operated a gaging station on the Floyd at James, Iowa (mile 10.7), since 1934. Daily flows of record are: maximum 71,500 c.f.s. (8 June 1953), mean 173 c.f.s., and minimum 0.9 c.f.s. (10-22 January 1977). The Corps of Engineers operated a daily suspended-sediment sample collection station at James from 16 March 1954 through 30 April 1957. Operation of this station was resumed by the USGS from 1 October 1968 through 30 September 1973. Daily suspended-sediment loads of record were: maximum 171,000 tons (25 May 1954), mean 1,230 tons, and minimum, no load (a number of days during water years 1956 and 1957). The maximum annual suspended-sediment load of record was 799,804 tons (water year 1971); however, the total suspended-sediment load for water year 1954 could possibly have exceeded the 1971 value based on the 6-1/2-month total of 729,587 tons. The average annual load was 449,561 tons. Average annual sediment yields range from 6,000 to 10,000 tons/square mile in the vicinity of Sioux City; however, yields decrease rapidly to 250-300 tons/square mile above James.

Improvements on the Floyd River at Sioux City were authorized by the Flood Control Act of 1958 (Public Law 85-500). This legislation provided for construction of a new 2-mile-long channel from the relocated

Missouri-Floyd confluence to the 18th Street Bridge, Figure 1; straightening and enlarging 4 miles of existing channel upstream from the 18th Street Bridge to the upper end of the project (north of 46th Street); construction of 9 miles of levee along both banks of the river; and construction of four new railroad bridges across the channel. The Public Law 85-500 appropriations were supplemented by a 1962 congressional authorization for twin bridges at the Interstate Highway 29 crossing, 300 ft upstream from the Missouri-Floyd confluence. Construction of the improvements on the Floyd at Sioux City began in June 1961 and was completed in July 1966. Total cost (1966) was $18,356,000, including $6,800,000 of non-Federal funds.

The Floyd River flood control project design included a high-velocity rock-lined channel to carry a maximum discharge of 71,500 c.f.s. (equal to the flood of record) below the level of the adjacent natural ground. The rock-lined channel would lie in the relatively erosion-resistant clays of the Missouri River floodplain: however, a few feet below the design bed grade were deep deposits of highly erodible sands and gravels. With the erodible material lying so close beneath the channel bottom, there was a high potential for deep localized scour and extensive bed degradation during high-velocity flows. In the event that extensive degradation occurred, it could cause serious damage by undermining bridge piers and abutments and the toes of revetted side slopes. Another consequence of degradation would be excessive drawdown of the water surface and destructive velocities in the proposed leveed earth channel upstream from the rock-lined reach. Therefore, it was considered necessary to place a series of grade stabilization structures that would deter head cutting in the event degradation occurred and could create sufficient head losses to maintain the design water-surface elevations. A brief model study at the University of California at Berkeley indicated that a row of sheet piling across the channel bed with some form of rock protection might adequately retard the development of head cutting and create sufficient losses to maintain the desired upstream water surface. Further

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model studies were conducted at the University of Iowa to develop criteria for adequate rock protection for the sheet piling.

The design of the lower mile of the project was dictated, to a large extent, by severe limitations on the available right-of-way and by the numerous streets and railroads crossing the project. The large number of crossings and the adjacent railroad yards required that the water-surface elevation be kept as low as possible in order to avoid the prohibitive cost of raising the elevation of the bridges and railroad yards. In addition, the bed grade of the realined channel had to meet the bed grade of the existing natural channel at the head of the reach, and at the same time the grade could not be so steep through the project reach that excessive velocities for discharges up to 71,500 c.f.s. would be produced. Normally these stringent requirements would dictate the use of a high-velocity concrete-lined channel; however, the cost of such a channel was prohibitive. The selected design called for a trapezoidal channel (with an erodible bed) and rock protected 1V-on-2.5H side slopes. The bottom width of the proposed channel would narrow from 280 ft at the Missouri River to 100 ft at the upper end of the project. Using this design, the average stream velocity through the reach would be slightly in excess of 14 fps for the design discharge of 71,500 c.f.s. Velocities would be greater than 10 fps for discharges more than 23,000 c.f.s., which is approximately the 25-year flood.

Computations based on the results of the model tests indicated that five sills spaced approximately 2,000 ft apart in the lower 1-1/2 miles of the project reach would effectively maintain desired water levels in the channel despite considerable bed scour; further, even if the bed was to degrade 10 ft, the water surface would be maintained within 1 to 2 ft of the design elevation with no degradation. As a result of these tests, five sheet piling and rock sills were placed at miles 0.1, 0.4, 0.8, 1.1, and 1.6 upstream from the new Missouri-Floyd confluence. These sills, completed in 1965, have been collectively designated as a Section 32

existing site. These structures provide a 4-ft/mile bed gradient through the improved channel (elevations through the project reach vary from approximately 1,070 ft at the Missouri-Floyd confluence to el 1,080 at mile 1.6).

The sharp-crested sills were constructed using single rows of sheet piling at the design bed elevation, Figures 2 and 3. The steel-sheet piling conformed to ASTM A-328-54, "Standard Specifications for Steel-Sheet Piling." The pile sections were required to be of the continuously interlocking type throughout their entire lengths when in place. The properties of the sections were specified as follows:

Type of Section	Nominal Web Thickness in.	Weight per Square Foot of Wall* lb
DA-27	3/8	27
SA-23	3/8	23

* Weight per square foot may not vary over 2.5 percent above or below the value shown.

The stone used to complete the structures was required to meet the following specification, Figure 3:

	Stone Size, lb		
	Maximum	Median	Minimum
Derrick stone	6,000	4,000*	2,000
36-in.-thick riprap	600	280	20
18-in -thick riprap	300	180	20
15- or 12-in.-thick riprap	150	80	20

* Half of the derrick stone by weight should be larger than this size.

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(Sheet 4 of 5)

Neither the breadth or thickness of any piece of stone was allowed to be less than one-third its length. The bedding material met the following gradation:

Sieve Size	Percent Passing by Weight
3 in.	60-100
3/4 in.	0-60
No. 4	0-30

Since the sills were constructed the new channel has experienced yearly maximum flows of 700 to 17,000 c.f.s. The peak flow, which was recorded in April 1979, was well under the design flow of 71,500 c.f.s. During an inspection on 20 March 1981, all five sills were visible and appeared to be in excellent shape, with the possible exception of the sill at station 2+90 which has been deformed in a downstream direction, Figure 4.

A survey made in April 1981 showed the sheet-piling of the sill at station 2+90 to be deformed, in a downstream direction, a maximum of 5 feet near the centerline. The sheet-pile at the area of the severest deformation was at an angle of 30^{o} from vertical and the downstream riprap had been displaced to a depth of approximately 6 feet immediately downstream of the sheet-piling.

The riprap downstream of sill No. 5 at station 83+00 has been displaced to a depth of 7 feet below the crest of the sheet-piling, Figures 6 and 7. There has been no deformation of the sheet-piling. The other three sills were in excellent condition and overall the project is functioning as designed.

ITEM 21-1
(Sheet 5 of 5)

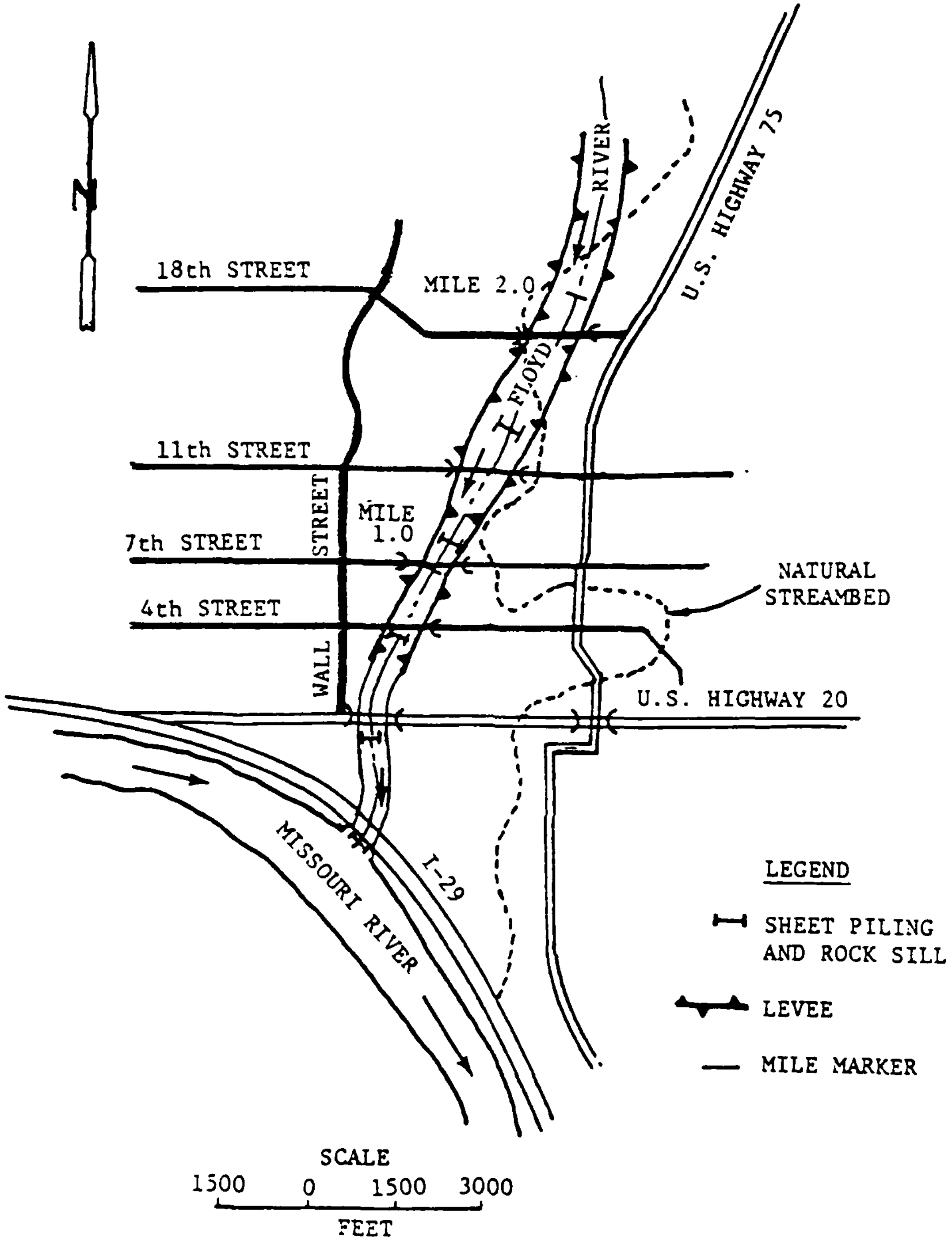


Figure 1. Floyd River at Sioux City, Iowa.

ITEM 21-2

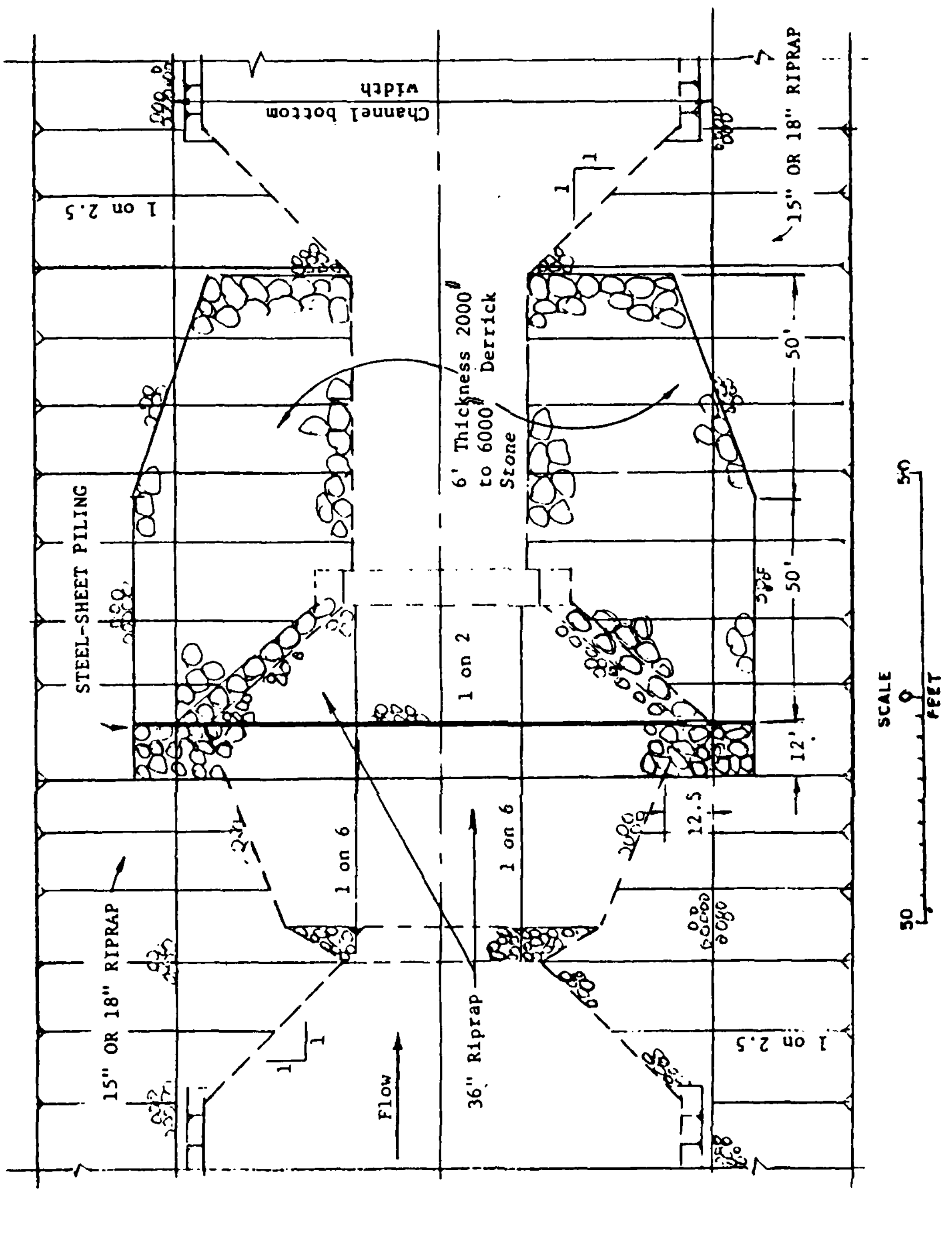
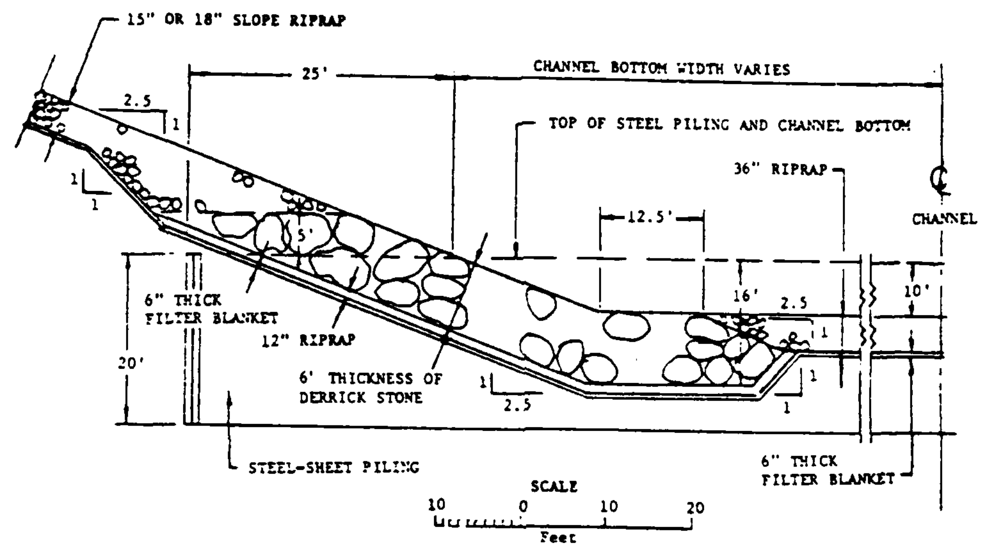
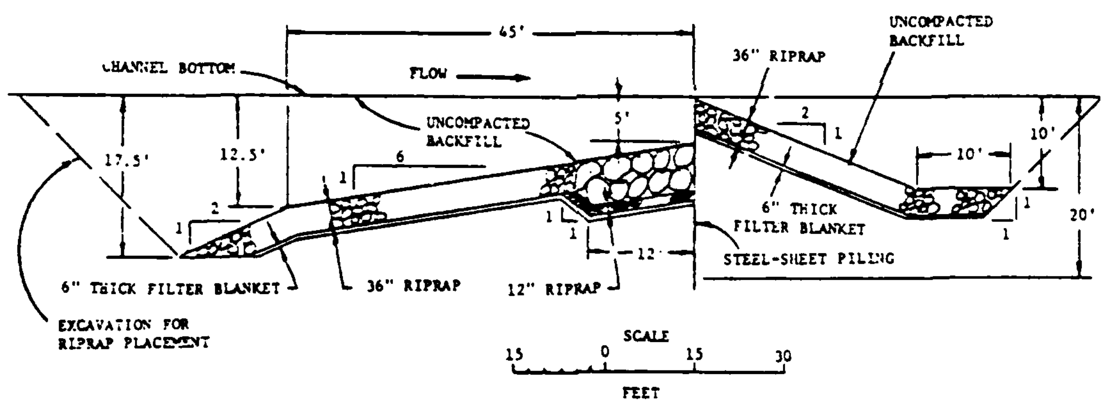


Figure 2. Floyd River at Sioux City, Iowa. Typical plan view of sheet piling and rock sill.



a. Half-sectional view of sheet piling and rock sill



b. Center-line view of sheet piling and rock sill

Figure 3. Floyd River at Sioux City, Iowa. Half-sectional and center-line view of sheet piling and rock sill.

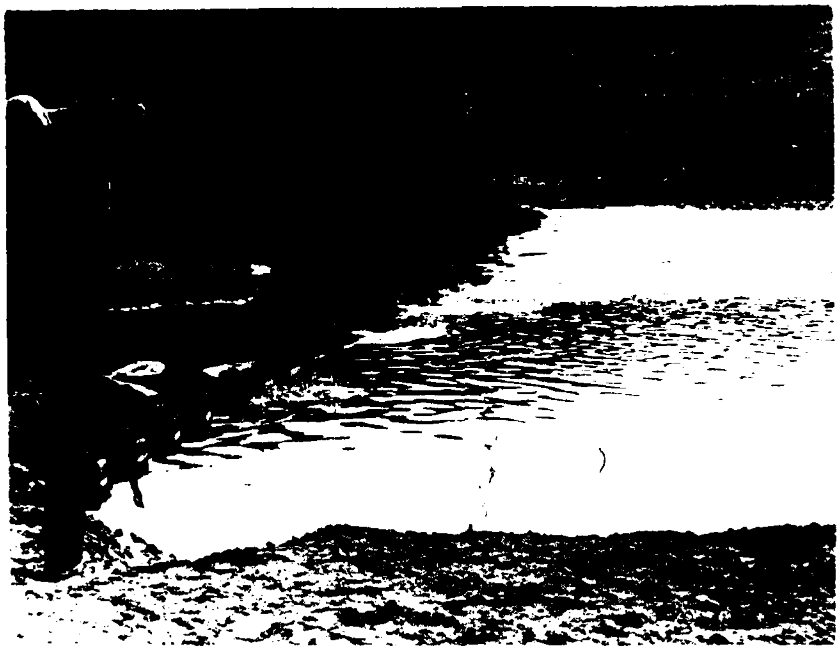


Figure 4. Floyd River at Sioux City, Iowa. Sill at Station 2+90. Sheet pile alinement has deformed in a downstream direction. Note flow of water concentrated in center. 20 March 1981.

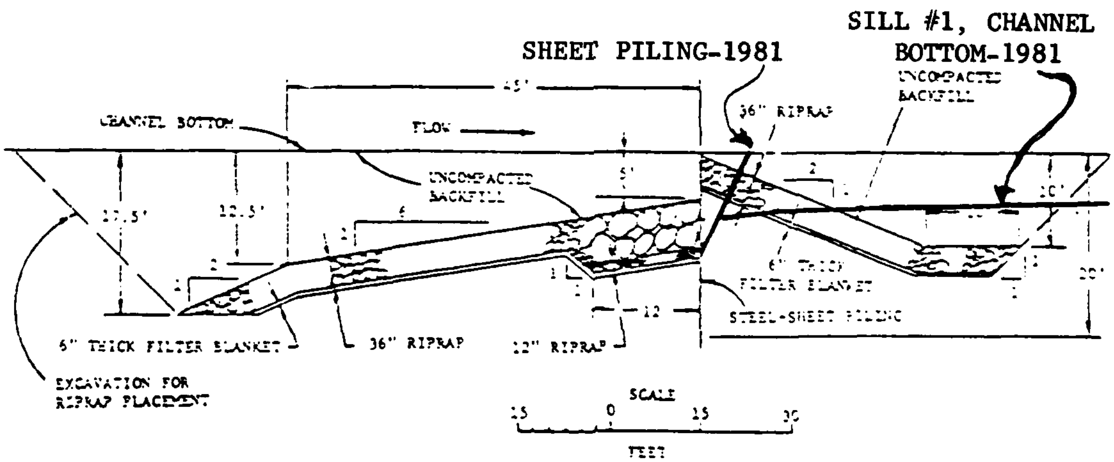


Figure 5. Floyd River at Sioux City, Iowa. Centerline view of sheet piling and rock sill at Sta. 2+90. Results of April 1981 survey superimposed.

ITEM 21-5

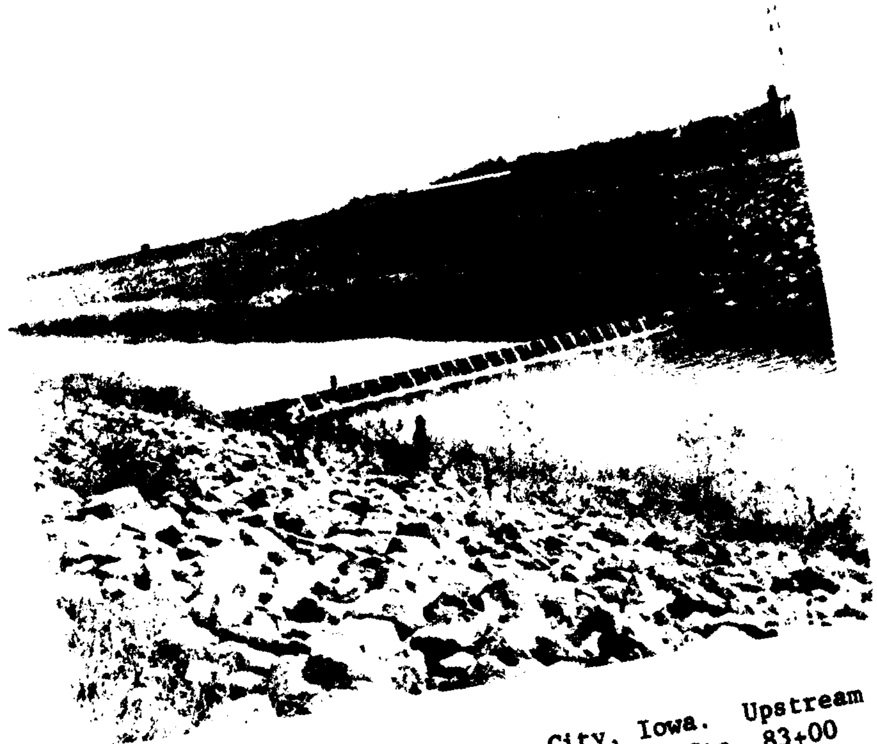


Figure 6. Floyd River at Sioux City, Iowa. Upstream view of sheet piling and rock sill #5, Sta. 83+00

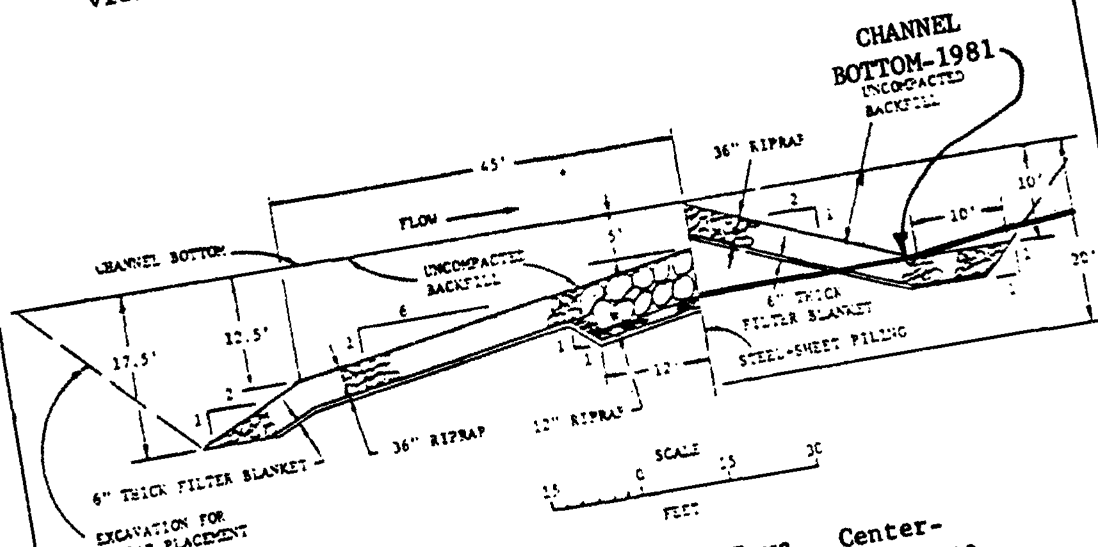


Figure 7. Floyd River at Sioux City, Iowa. Center-line view of sheet piling and rock sill #5, Sta. 83+00 Results of April 1981 survey superimposed.

WEST FORK DITCH
ONAWA, IOWA

Streambank Erosion Control Evaluation and Demonstration Act of 1974
Section 32 Program - Work Unit 2

EVALUATION OF EXISTING BANK PROTECTION WORKS

(1) Location

Stream West Fork Ditch River Mile ______ Side ______

Local Vicinity ______ Lat N42°0' Long W96°10'

At/Nr City Onawa County Monona State IA Cong Dist 6

CE Office Symbol MRO Responsible Agency Corps of Engineers

Site Map Sources Corps of Engineers Omaha District

Land Use Information Farmland and small communities

(2) Hydrology at or Near Site

Stage Range ______ to ______ ft; Period of Record 1939 to 1981.

Discharge Range 0.2 to 12,400 cfs; Velocity Range ______ to 8 fps

Sediment Range 0 to 204,000 tpd; Period of Record 1957 to 1967.

Bank-full Stage ______ ft; Flow ______ cfs; Average Recurrence Interval ______ yr

Bank-full Flow Velocity: Average ______ fps; Near Bank ______ fps

Comments ______

(3) Geology and Soil Properties

Bank (USCS) Fat clays, (CH) clayey silt (ML), lean clay (CL) Bed (USCS) Same

Data Sources Corps of Engineers test borings

Groundwater Bank Seepage High water table causes significant bank instability during wet periods.

Overbank Drainage ______

Comments ______

(4) Construction of Protection

Need for Protection Channel degrading posing a threat to adjacent levees.

Erosion Causative Agents Straightened channel resulting in steepened gradient and increased velocity.

Protection Techniques Five low rock sills with sheet pile added later.

General Design Originally graded stone placed on filter fabric.

Project Length 48,500 ft; Construction Cost $54,150 Mo/Yr Completed 1972

(5) Maintenance

Experienced Flows (Stage, cfs, Date)

Repairs and Costs (Item, Cost, Data) Sill 2 removed $52,000 1973. Sills 1,3,4,& 5 modified with 2 row sheet pile and riprap, $91,000, 1973.

Comments:

(6) Performance Observations and Summary

Monitoring Program Visual inspections and topographical surveys.

Documentation Sources Omaha District, Corps of Engineers

Project Effect on Stream Regime Stabilized channel.

Project Effect on Environment Negligible

Successful Aspects Has stopped channel degradation.

Unsuccessful Aspects Lateral erosion downstream of sills threatened to fail adjacent levees.

General Evaluation Overall project performing well.

Recommendations Preformed scour hole should be riprapped at time of intial contruction.

(7) Additional Information, Comments, and Summary

Map No. 22.

Attached Items:

22-1 Project summary
22-2 Project location
22-3 Plan view-original
22-4 Cross section and photograph
22-5 Plan view-modification
22-6 Cross section-modification

West Fork Ditch at Onawa, Iowa (Mile 0.0 to 9.2)

The Little Sioux River Basin Flood Control Project was authorized by the Flood Control Act of 1947 as modified in 1954. The project was designed to protect 188,000 acres of highly productive farmland and several small communities located along the Little Sioux River from its mouth to the community of Smithland, Iowa. Construction began in 1956 and was finished in 1966. The completed project provided for placement of 138 miles of levee and 62 miles of channel improvements which included the enlargement and straightening of existing channels or the excavation of new channels. Since 1959, when the project became partially operational, it has prevented flood damages estimated at more than $19,580,000. The total cost of the project was $18,483,000 (1972), including $3,000,000 derived from non-Federal sources. Local interests operate and maintain the project.

West Fork Ditch was dug by the Monona-Harrison Drainage District in the early 1900's. This ditch now drains the watershed previously emptied by the West Fork of the Little Sioux. Flow from the natural channel of the West Fork is diverted into the ditch near Holly Springs, Iowa. The West Fork Ditch discharges at the intersection of Wolf Creek, the Garretson Outlet Ditch, the Monona-Harrison Ditch, and the Little Sioux/Monona-Harrison Diversion Channel (Figure 1). When the western portion of the Little Sioux Basin is in flood, the discharges of West Fork Ditch, Garretson Outlet Ditch, and Wolf Creek are passed into the Monona-Harrison Ditch via the diversion channel. Because the runoff characteristics of the basin differ at opposite ends of the diversion channel, rarely do flood crests occur simultaneously. Generally, the crest in the western part of the basin occurs much earlier than in the eastern portion.

The USGS operated a gaging station on the West Fork of the Little Sioux at Holly Springs, Iowa (mile 11.4), from April 1939 through September 1969. The station was relocated on the Iowa State Highway 141 Bridge

at Hornick, Iowa (mile 9.2) in July 1974 with data collection continuing to the present, Figure 1. The daily discharges of record are maximum 12,400 c.f.s., mean 94.3 c.f.s., and minimum 0.2 c.f.s. The maximum observed flow velocity has been 8 fps. The discharges measured at the gaging station are considered typical of the West Fork project reach, although some additional discharge is accepted from Farmers Ditch at mile 6.9. MRO operated a suspended-sediment sample collection station at Holly Springs from August 1957 through September 1967. Daily suspended-sediment loads of record were: maximum 204,000 tons (14 June 1967), mean 1,178 tons, and minimum zero (on several days). The maximum annual load of record was 911,200 tons (water year 1962); the average annual load was 429,854 tons. Average annual sediment yields in this region are 6,000 to 10,000 tons/square mile, which are among the highest yields in the entire Mississippi River Basin. The surface soils through the project reach are a mixture of fat clays (CH), clayey silts (ML), and lean clays (CL).

The West Fork levee system was completed in June 1964. By the late 1960's channel degradation had become a serious threat to the levees and several internal drainage structures. Five low rock sills were placed in 1971-1972 at approximately equal intervals along the channel (at headcut locations) between mile 0.0 and the Iowa State Highway 146 Bridge at Hornick, Figure 1. The sills were constructed from rock placed on filter fabric. The fabric was specified to be monofilament yarn woven in a rectangular pattern; however, the type of fabric used is not known. The sills were constructed as a weir set at the design channel elevation, with a downstream apron 3 ft below the weir crest; riprap was placed along the channel side slopes for an additional 40 ft downstream from the apron, Figures 2 and 3. The stone used to construct the low-rock sills was specified to be 24 in. in diameter (maximum dimension) with the following gradation:

Weight, lb	Percent Lighter by Weight
1,500-2,500	100
1,000-1,800	50
500-1,000	15

ITEM 22-1
(Sheet 2 of 3)

The completed project provided for a bed-gradient of 1.5 ft/mile for a design flow of 11,100 c.f.s. The total cost (1972) for placement of the five sills was $54,150.

High flows of long duration in early 1973 produced severe lateral erosion downstream from the sills which in turn threatened to fail the adjacent levees; in addition a county bridge below sill 2 was seriously threatened. Emergency work was required in April 1973. Setback levees were constructed at sills 3 and 4, and sill 2 was removed ($52,000). In October 1973, the remaining sills were modified according to the results of limited model studies conducted at Mead Hydraulic Laboratory. The modification included two rows of sheet piling driven to refusal in the weir section to provide positive (uniform) cross-channel flow, Figures 4 and 6. In addition, the modification called for placement of riprap and shaping the downstream channel side slopes according to the recommendations of the model studies, Figure 5. The sheet piling complied with the guidelines provided by ASTM A 328-70, "Steel-Sheet Piling." The stone riprap used for the repairs had the same specifications as the stone used for construction of the original sills. The total cost of these modifications was $91,000 (1973). The four modified sills were designated as existing Section 32 sites.

Topographic surveys conducted since 1973 indicate that the channel is in equilibrium. No headcutting has been observed and the four modified sill structures are in an as-built condition. At the date of the last inspection visit in March 1981, the project was performing well, Figure 3.

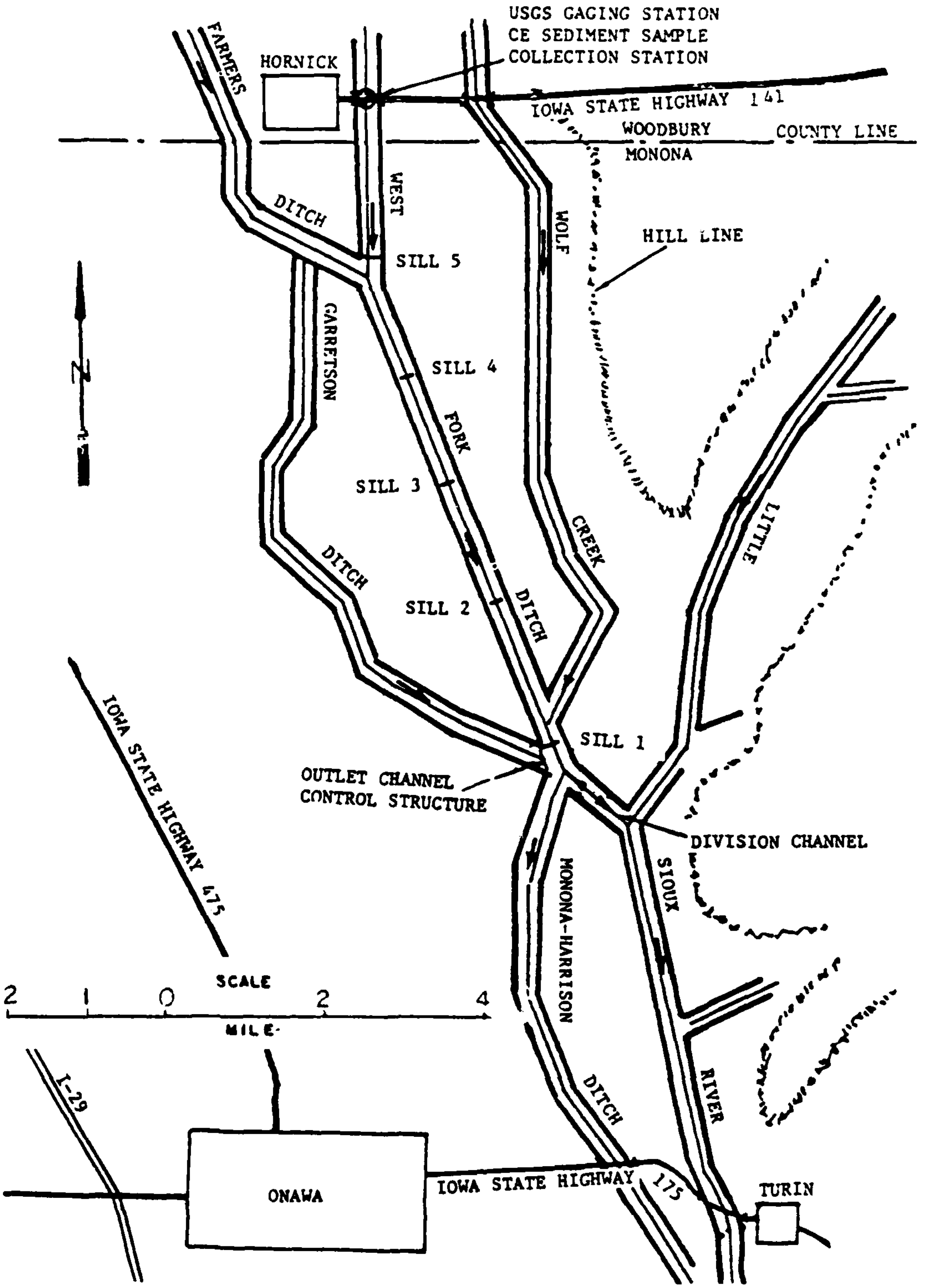
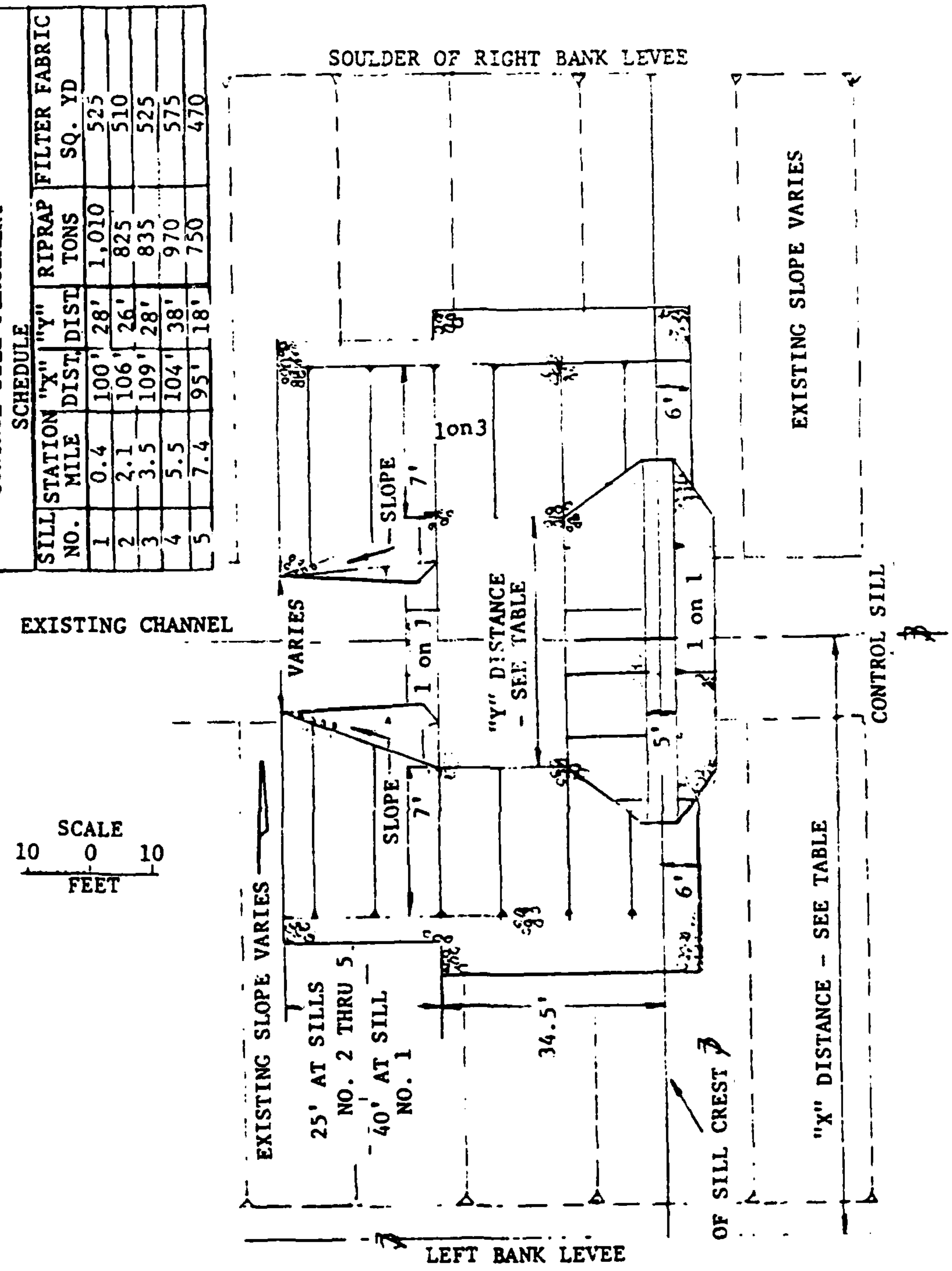


Figure 1. West Fork Ditch at Onawa, Iowa. Heavy lines paralleling streams represent levees.

ITEM 22-2



CONTROL SILL PLACEMENT SCHEDULE

SILL NO.	STATION MILE	"X" DIST.	"Y" DIST	RIPRAP TONS	FILTER FABRIC SQ. YD
1	0.4	100'	28'	1,010	525
2	2.1	106'	26'	825	510
3	3.5	109'	28'	835	525
4	5.5	104'	38'	970	575
5	7.4	95'	18'	750	470

Figure 2. West Fork Ditch at Onawa, Iowa. Plan view of low-rock sills.

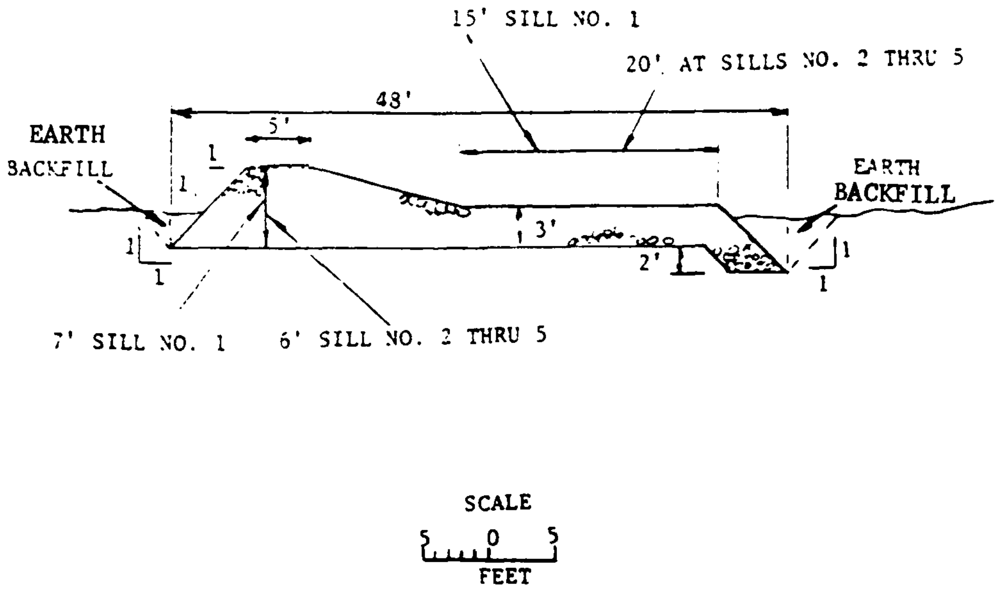


Figure 3. West Fork Ditch at Onawa, Iowa. Cross-sectional view (through structure center line) of low-rock sills

Figure 4. West Fork Ditch at Onawa, Iowa. Sheet-pile crest at sill 3 (March 1981)

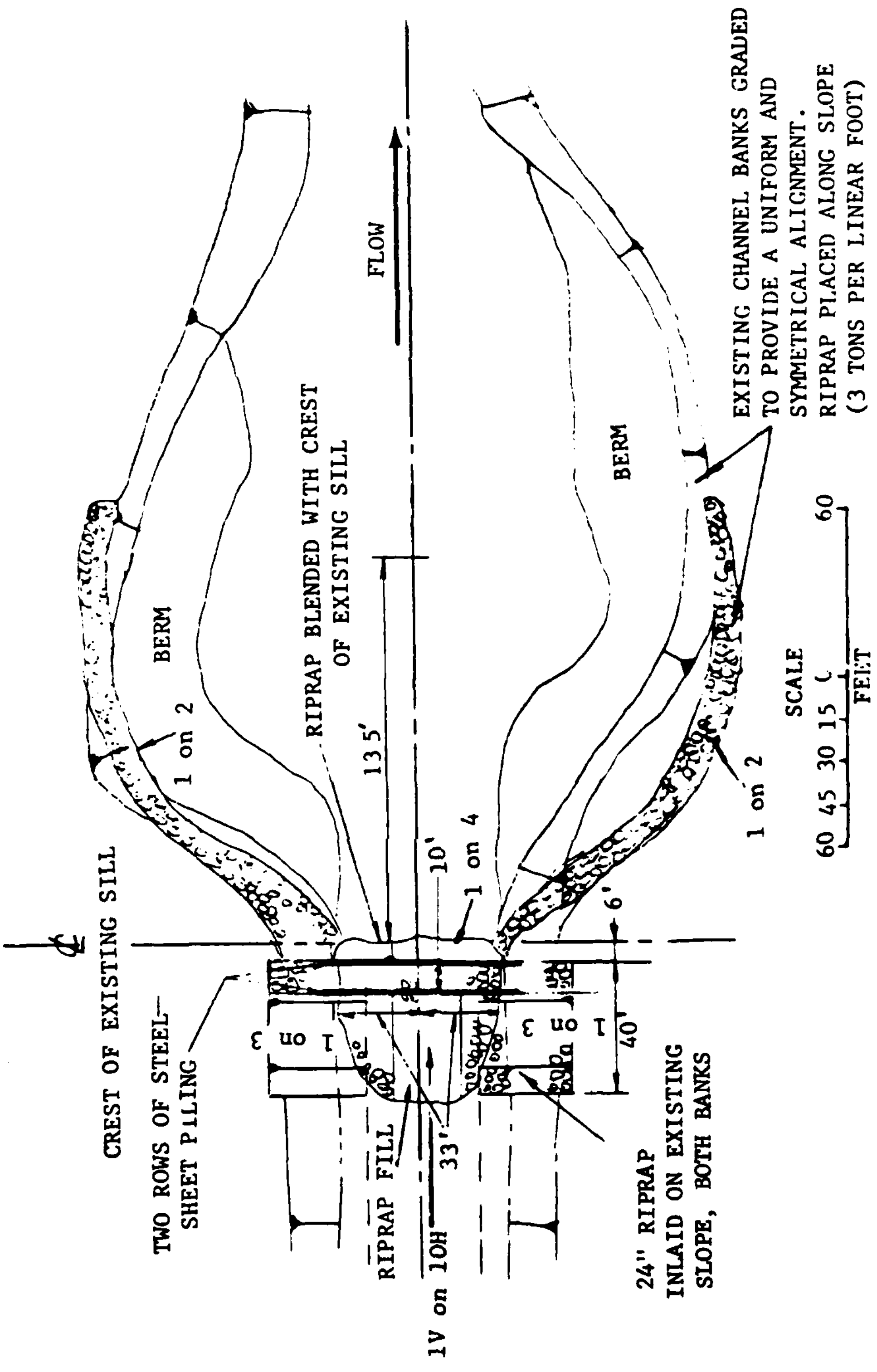


Figure 5. West Fork Ditch at Onawa, Iowa. Plan view of 1973 sill modification.

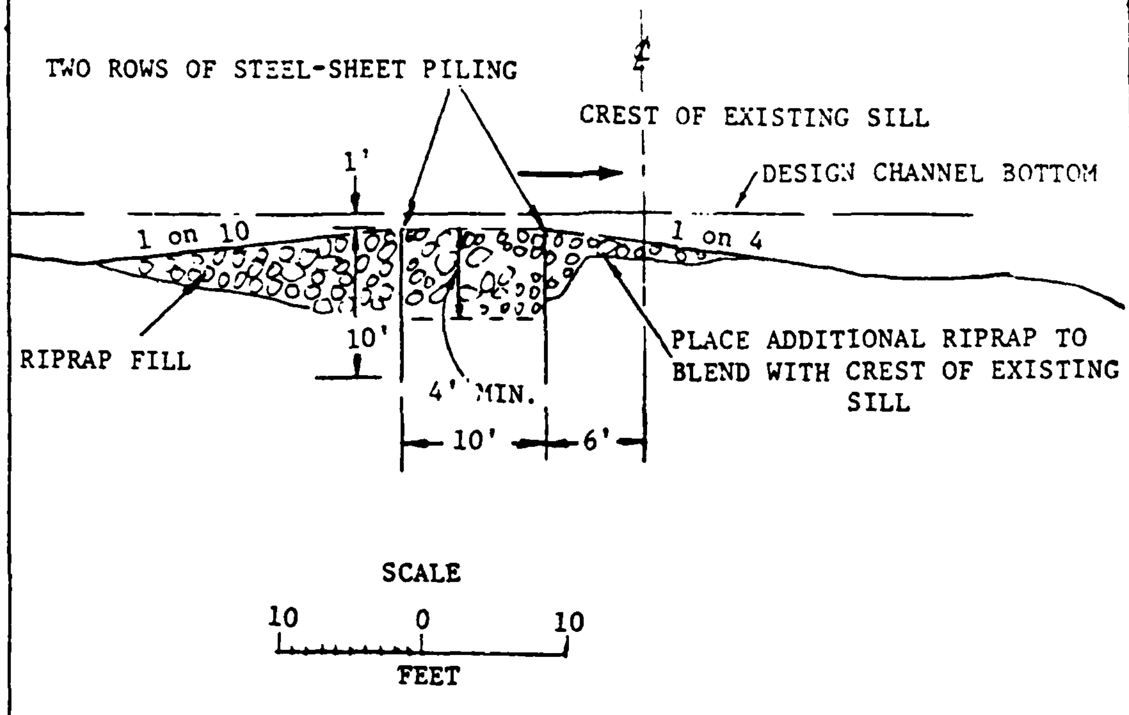


Figure 6. West Fork Ditch at Onawa, Iowa. Cross-sectional view (structure center line) of 1973 sill modification

CONNECTICUT RIVER
HANOVER, NEW HAMPSHIRE

Streambank Erosion Control Evaluation and Demonstration Act of 1974
Section 32 Program - Work Unit 2

EVALUATION OF EXISTING BANK PROTECTION WORKS

(1) Location

Stream Connecticut River River Mile 219.2-221.2 Side Left

Local Vicinity Hanover, NH Lat N43°42.5' Long W72°17.5'

At/Nr City Hanover County Grafton State NH Cong Dist 2

CE Office Symbol NED Responsible Agency New England Power Company

Site Map Sources USGS Topographic Quadrangle for Hanover, VT-NH, 1959

Land Use Information Source Dartmouth College

(2) Hydrology at or Near Site

Stage Range 380 to 385 ft; Period of Record 1949 to 1980.

Discharge Range very small to 50,000 cfs; Velocity Range very small to 3-10 fps

Sediment Range ukn to - tpd; Period of Record 19 - to 19 -.

Bank-full Stage ukn ft; Flow ukn cfs; Average Recurrence Interval ukn yr

Bank-full Flow Velocity: Average ukn fps; Near Bank ukn fps

Comments Site located in pool behind Wilder Dam a "run of river" facility Site is subject to daily water-surface fluctuations of about 2 feet.

(3) Geology and Soil Properties

Bank (USCS) Sandy silt to silt med/fine sand Bed (USCS) Sandy silt to med/fine sand

Data Sources Corps of Engineers

Groundwater Bank Seepage None observed

Overbank Drainage None observed

Comments

(4) Construction of Protection

Need for Protection To prevent streambank erosion.

Erosion Causative Agents Steep slopes, velocity, pool and groundwater fluctuations, wave action, freeze-thaw, and ice action.

Protection Techniques Riprap revetment

General Design Riprap layer overlying gravel fill and sand bedding material.

Project Length 9,000 ft; Construction Cost $ * Mo/Yr Completed 1962*

* See attached Item 23-1 for above information.

(5) Maintenance

Experienced Flows (Stage, cfs, Date) 385 ft NGVD, 50,000 cfs, 1 July 1973

Repairs and Costs (Item, Cost, Data) None to date

Comments:

(6) Performance Observations and Summary

Monitoring Program Semiannual (spring and fall)

Documentation Sources Photographs and trip reports for each visit

Project Effect on Stream Regime Negligible

Project Effect on Environment Negligible

Successful Aspects Project generally good condition with no signs of erosion.

Unsuccessful Aspects Minor erosion of upper slope by surface runoff.

General Evaluation Toe protection in good condition. Upper bank not protected from surface runoff.

Recommendations Surface runoff should be controlled and vegetation established on slope.

(7) Additional Information, Comments, and Summary

Map No. 23. In more than 20 yrs that the revetment has been in place it has generally stood up well.

Attached Items:

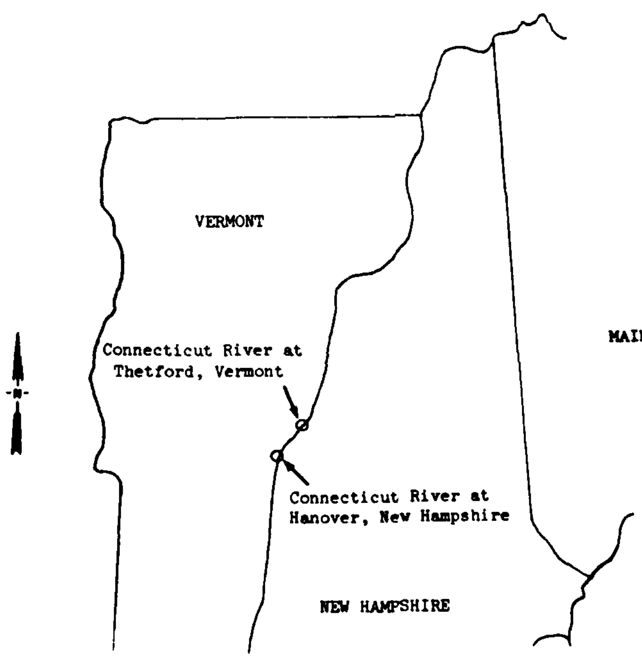
23-1 Project summary and location
23-2 Project vicinity map
23-3 Cross section and general view
23-4 Photographs immediately and 26 yrs after construction

Connecticut River

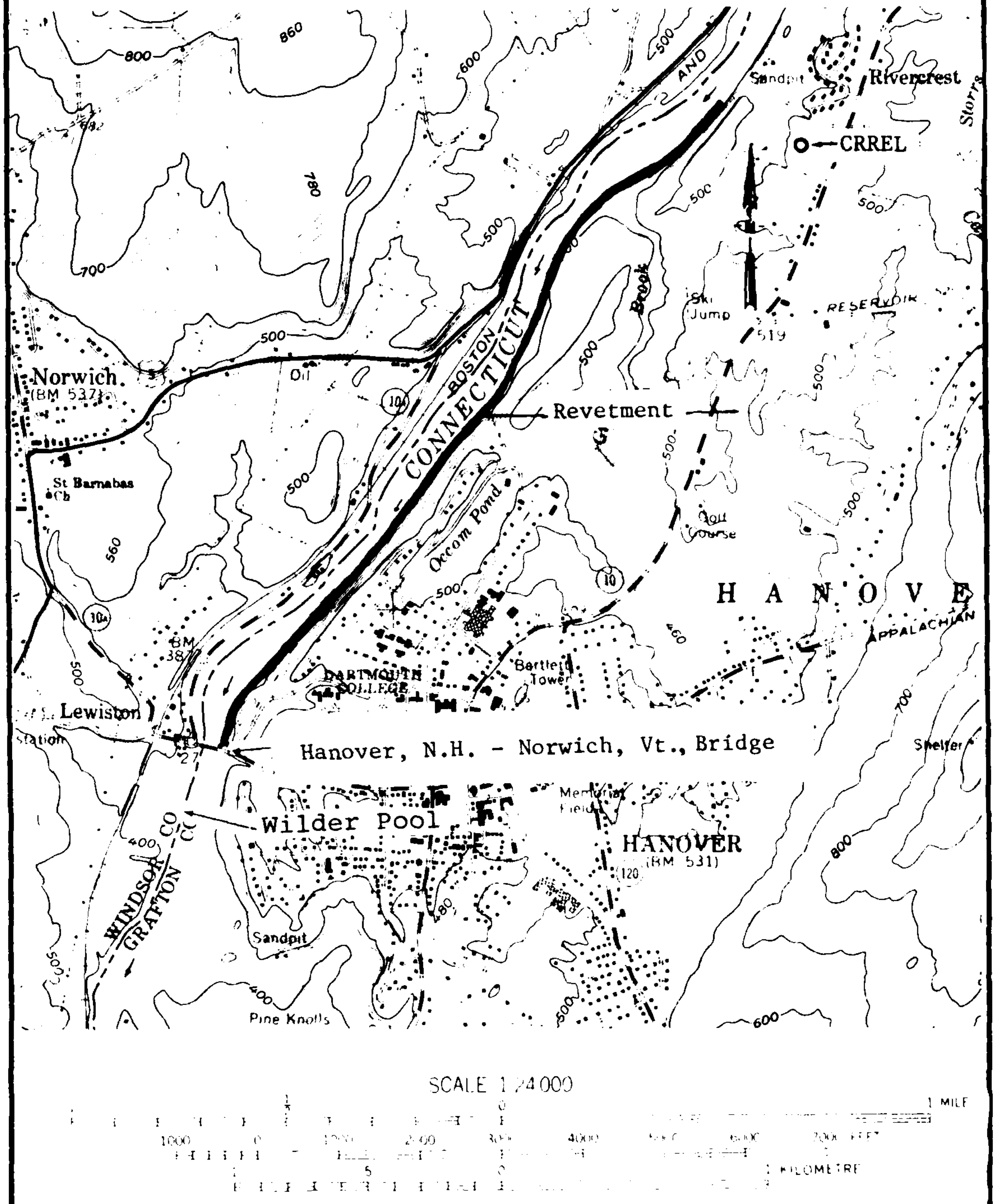
at Hanover, N.H.

Between 1950 and 1962, New England Power Co. (NEPCO) placed "run-of-the-blast" stone riprap along 9000 ft. of the left bank of the Connecticut River that fronts Dartmouth College. NEPCO followed no job specification except that it did not accept any stone having diameters larger than 12 in. The stone was placed from a barge. The site is subject to daily power pool fluctuations of approximately 2 ft. At the time of the 1978 and 1979 inspections, a stand of vegetation had become established, and the revetment was performing adequately. The NEPCO Engineering Department indicates that only minor failures have occurred and no major repairs have been necessary. There are a few small areas of erosion above the stone riprap where some large trees have become unstable and fallen taking part of the bank with them. These trees are above the elevation of any recently recorded flood levels.

*Construction	4800± ft.	$132,000	1953
Phased over several	2150	$24,800	1956
years	2150	$37,500	1962

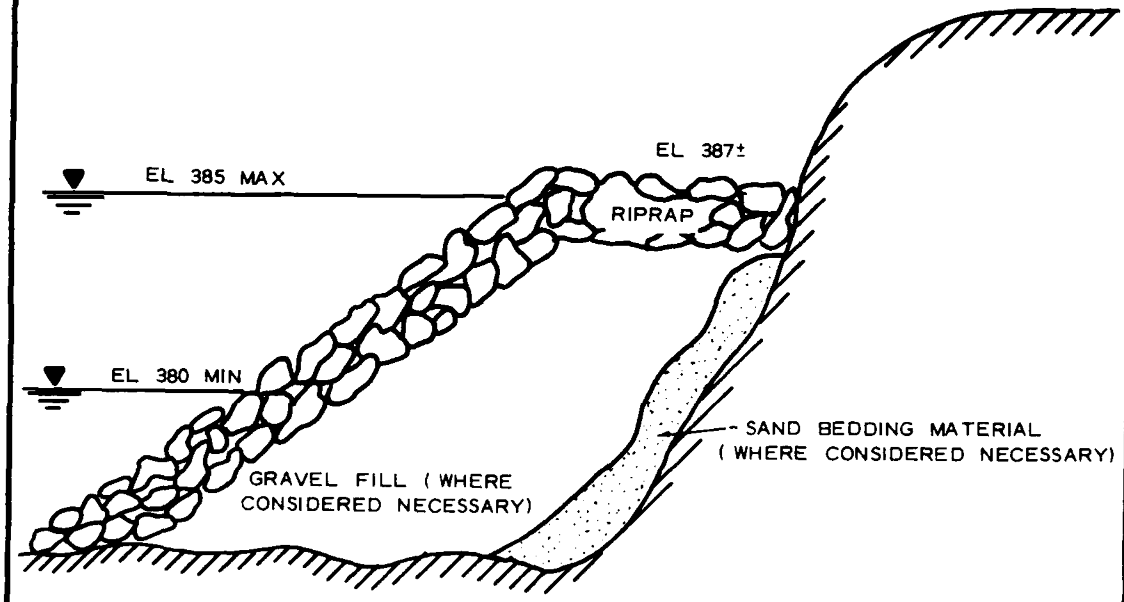


ITEM 23-1

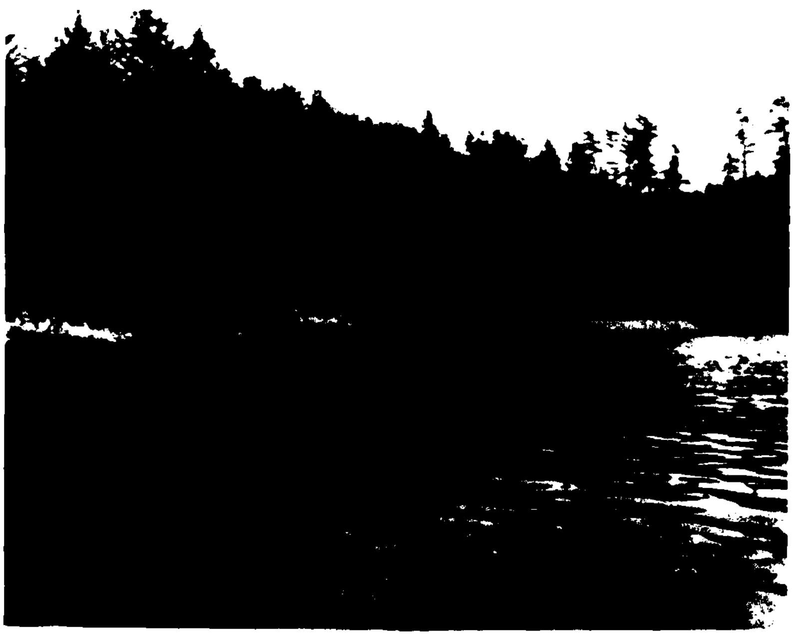


CONNECTICUT RIVER AT HANOVER, N. H.
(Source: USGS 1:24,000 topographic quadrangle map for Hanover, Vt.-N. H., 1959)

ITEM 23-2



Cross-sectional plan of riprap revetment, Connecticut River at Hanover, N. H. (diagram furnished by New England Division)



View of sand bedding material, Connecticut River at Hanover, N. H. The revetment work was built as part of the Wilder Dam reconstruction project. The normal high water is now within 1 ft of the top of the revetment (photograph furnished by New England Power Company, 1954)

View of in-place riprap, Connecticut River at Hanover, N. H. The revetment work was built as part of the Wilder Dam reconstruction project. The normal high water is now within 1 ft of the top of the revetment. Streamflow is to the right. (Photograph furnished by New England Power Service Co., 1954)

View of riprap 26 years after construction on Connecticut River at Hanover, N. H.

PHOTOGRAPHS IMMEDIATELY AND 26 YEARS AFTER CONSTRUCTION

ITEM 23-4

CONNECTICUT RIVER
THETFORD, VERMONT

Streambank Erosion Control Evaluation and Demonstration Act of 1974
Section 32 Program - Work Unit 2

EVALUATION OF EXISTING BANK PROTECTION WORKS

(1) Location

Stream Connecticut River River Mile apr 225 Side Right

Local Vicinity Thetford, VT Lat N43° 45' Long W72°13'

At/Nr City Thetford County Orange State VT Cong Dist at large

CE Office Symbol NED Responsible Agency Mr. Lyman Allan

Site Map Sources USGS Topographic quadrangle map for Mt. Cube, NH-VT, 1931

Land Use Information Sources Private home resident (Lyman Allan)

(2) Hydrology at or Near Site

Stage Range 380 to 386 ft; Period of Record 1949 to 1980.

Discharge Range v. small to 50,000 cfs; Velocity Range v. small to 3-10 fps

Sediment Range ukn to - tpd; Period of Record 19 - to 19 -.

Bank-full Stage ukn ft; Flow ukn cfs; Average Recurrence Interval ukn yr

Bank-full Flow Velocity: Average ukn fps; Near Bank ukn fps

Comments Site located in pool behind Wilder Dam a "run of river" hydro facility. Site is subject to daily water-surface fluctuations of about 2 feet.

(3) Geology and Soil Properties

Bank (USCS) Sandy silt to silty med/fine sand Bed (USCS) Sandy silt to med/fine sand

Data Sources Corps of Engineers

Groundwater Bank Seepage None observed

Overbank Drainage None observed

Comments

(4) Construction of Protection

Need for Protection To prevent streambank erosion

Erosion Causative Agents Steep slopes, river velocities, pool and groundwater fluctuations, wave action, frost and freeze-thaw action on cohesionless soil, ice action

Protection Techniques Used-automobile tire retaining wall

General Design Revetment consists of used tires fill with stone. Tires were unattached and laid horizontally in a stepped manner back from the toe of bank

Project Length 150 ft; Construction Cost $ 0 landowner used free materials Mo/Yr Completed 1972

(5) Maintenance

Experienced Flows (Stage, cfs, Date) 386 ft NGVD. 50,000 cfs, 1 July 1973 (est)

Repairs and Costs (Item, Cost, Data) Initially tires were realigned annually, However, have since stabilized and only require minor maintenance, usually in spring.

Comments: Material cost minimal. Construction and repairs time-consuming

(6) Performance Observations and Summary

Monitoring Program Semiannual (spring and fall)

Documentation Sources Photographs and trip reports

Project Effect on Stream Regime Negligible

Project Effect on Environment Negligible

Successful Aspects Revetment has remained in place and there has been no significant erosion. Revetment was low cost and has worked very well

Unsuccessful Aspects None

General Evaluation No signs of significant erosion were observed during visits Project in generally good condition.

Recommendations This form of revetment is ideal as a self-help measure which a landowner can undertake on his own.

(7) Additional Information, Comments, and Summary

Map No. 24. Since placement costs are high this form of protection is best undertaken by a landowners own available labor resources.

Attached Items.

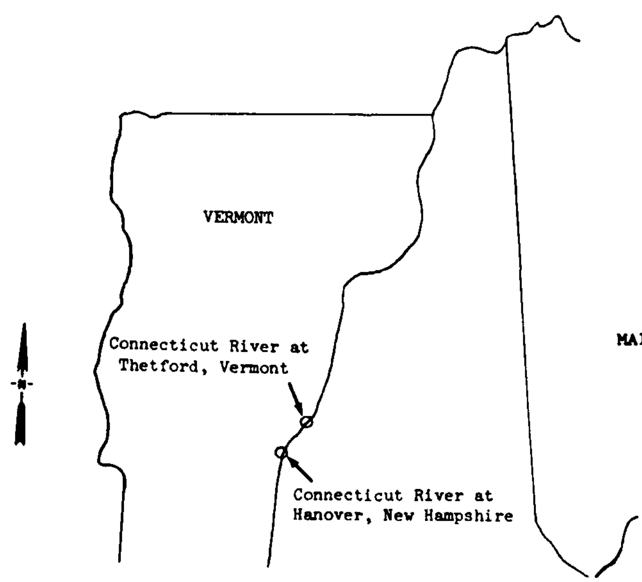
24 - 1 - Project summary and location

24 - 2 - Initial work and site map
24 - 3 - Plan and cross section
24 - 4 - Low and Normal pools
24 - 5 - Initial inspection photographs
24 - 6 - Final inspection Photographs

Connecticut River

at Thetford, Vermont

In summer 1972, private landowners constructed a 150-ft-long used-tire revetment on the right bank of the Connecticut River. Height of the protection ranged from three to fourteen layers of tires. Each tire was filled with small stones and tamped. At the time of the 1978 inspection, the revetment was intact, and a good stand of vegetation had become established on the upper bank. Minor structural damage was noted in a few places, including undercutting of tires at the upstream end of the revetment, which has resulted in the sagging of some tires and the spilling of fill material. There has also been some displacement of tires due to impact of ice cakes and heavy overbank drainage. Additional tires have been added to maintain the same top elevation. The New England Division, CE, has recommended that holes be drilled in the sidewalls to prevent flotation, that tires be tied together and anchored to the bank, that tires be stepped back 6 to 12 inches with each row, and that the space between the tires and bank be filled with earth or stone to prevent water or ice flow behind the structure.

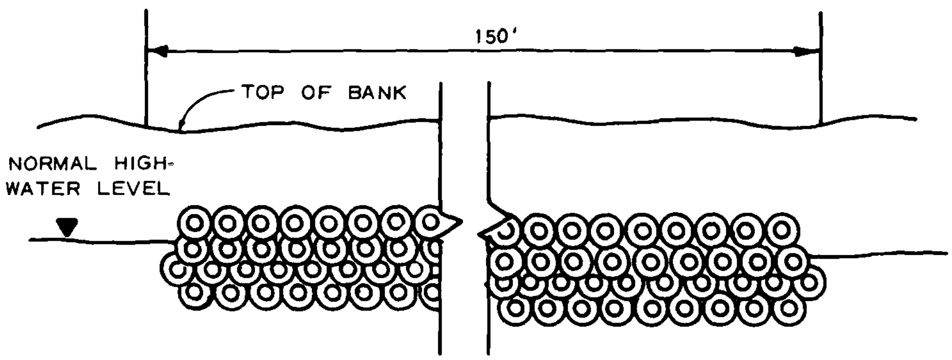


ITEM 24-1

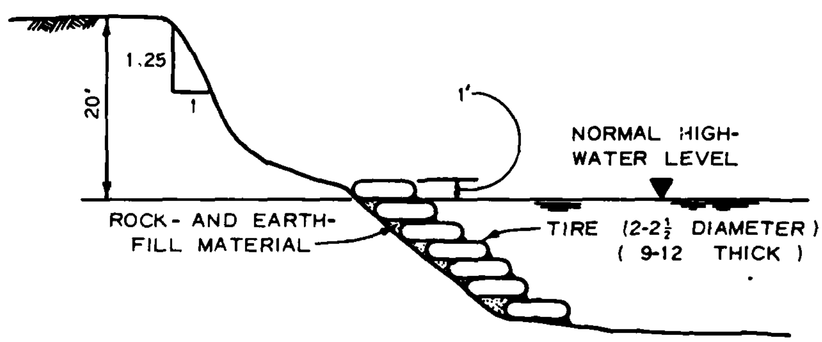
Initiation of used-automobile tire placement in the spring of 1972, Connecticut River at Thetford, Vt. Note eroded bank in background. The Allen residence is located approximately 50 ft landward from the top bank. View is upstream

Connecticut River at Thetford, Vt. (Source: USGS 1:62,500 topographic quadrangle map for Mt. Cube, N. H.-Vt., 1931)

ITEM 24-2



PLAN VIEW



CROSS SECTION

Plan and cross-sectional views of used-automobile tire revetment, Connecticut River at Thetford, Vt. (diagram furnished by New England Division)

ITEM 24-3

ITEM 24-4

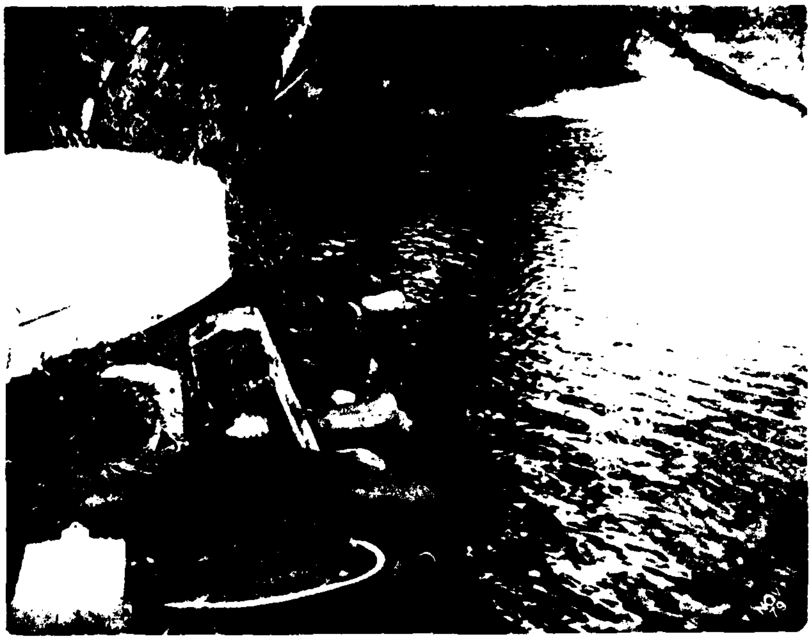
Completed revetment shown at low water, Connecticut River at Thetford, Vt. View is upstream

At the time of the inspection visit, vegetation had become well established above the revetment, Connecticut River at Thetford, Vt. View is downstream ([illegible])

The majority of the used automobile tires have remained in place, although minor realignment has been required, Connecticut River at Thetford, Vt. View is upstream (26 July 1978)

Stereoscopic view of used automobile tire revetment taken during inspection visit, Connecticut River at Thetford, Vt. (26 July 1978)

ITEM 24-5



Upstream view of used automobile tire revetment of right bank of the Connecticut River at Thetford, VT. 1980

Downstream view of used automobile tire revetment on right bank of the Connecticut River at Thetford, VT. 1980

ITEM 24-6

CONNECTICUT RIVER
TURNERS FALLS POOL, MASSACHUSETTS

Streambank Erosion Control Evaluation and Demonstration Act of 1974
Section 32 Program - Work Unit 2

EVALUATION OF EXISTING BANK PROTECTION WORKS

(1) Location

Stream Connecticut River River Mile 126.4-136.1 Side Both

Local Vicinity Turner's Falls Pool Lat N42°37' Long W72°29'

At/Nr City Northfield County Franklin State MA Cong Dist 1

CE Office Symbol NED Responsible Agency Northeast Utilities

Site Map Sources USGS topographic quadrangle maps for Northfield Turners Falls, MA 1976

Land Use Information Sources Northeast Utilities

(2) Hydrology at or Near Site

Stage Range 175 to 216 ft; Period of Record 1915 to 1980.

Discharge Range very small to 210,000 cfs; Velocity Range very small to 3-5 fps

Sediment Range ukn to --- tpd; Period of Record 19 – to 19 –.

Bank-full Stage ukn ft; Flow ukn cfs; Average Recurrence Interval ukn yr

Bank-full Flow Velocity: Average ukn fps; Near Bank ukn fps

Comments Site is located behind Turners Falls Dam a "run of river" hydro facility. Site is subject to daily water-surface fluctuations of about 3.5 feet.

(3) Geology and Soil Properties

Bank (USCS) Sandy silt to medium/fine sand Bed (USCS) Sandy silt to med/fine sand

Data Sources Corps of Engineers

Groundwater Bank Seepage None observed

Overbank Drainage Observed at several locations in Turners Falls Pool

Comments

(4) Construction of Protection

Need for Protection To prevent streambank erosion

Erosion Causative Agents Steep slopes, pool & groundwater fluctuations, wave action, river velocity, freeze-thaw on cohesionless soil, ice attack.

Protection Techniques Tree clearing, riprapping, and hydroseeding.

General Design Trees were cut then removed by helicopter. A slurry consisting of a mixture of grass and shrub seeds was applied. Selected toe areas were riprapped

Project Length 9 miles ft; Construction Cost $ * Mo/Yr Completed 1978*

* See attached Item 25-1 for above information.

(5) Maintenance

Experienced Flows (Stage, cfs, Date) 194 ft NGVD, 82,400 cfs, 15 March 1977

Repairs and Costs (Item, Cost, Data) None to date

Comments

(6) Performance Observations and Summary

Monitoring Program Semiannual inspections (spring and fall)

Documentation Sources Photographs and trip reports

Project Effect on Stream Regime Negligible

Project Effect on Environment Has restored vegetative cover to major sections of banks. Many trees lining the bank have been removed.

Successful Aspects Areas with riprap are intact with no signs of erosion to date.

Unsuccessful Aspects Areas involving tree removal and hydroseeding without riprap on steep slopes do not appear too successful. Better success on flatter natural slopes

General Evaluation Areas involving tree removal and hydroseeding alone are in poor to fair condition. Riprap revetment was in good condition with no erosion.

Recommendations Riprapped reaches working quite well. Hydroseeding of natural banks alone is only a marginal short-term solution.

(7) Additional Information, Comments, and Summary

Map No. 25. The work was being conducted by Northeast Utilities. The work will be conducted along several reaches of the river.

Attached Items:

25-1 Project summary and location

25-2 Project vicinity map

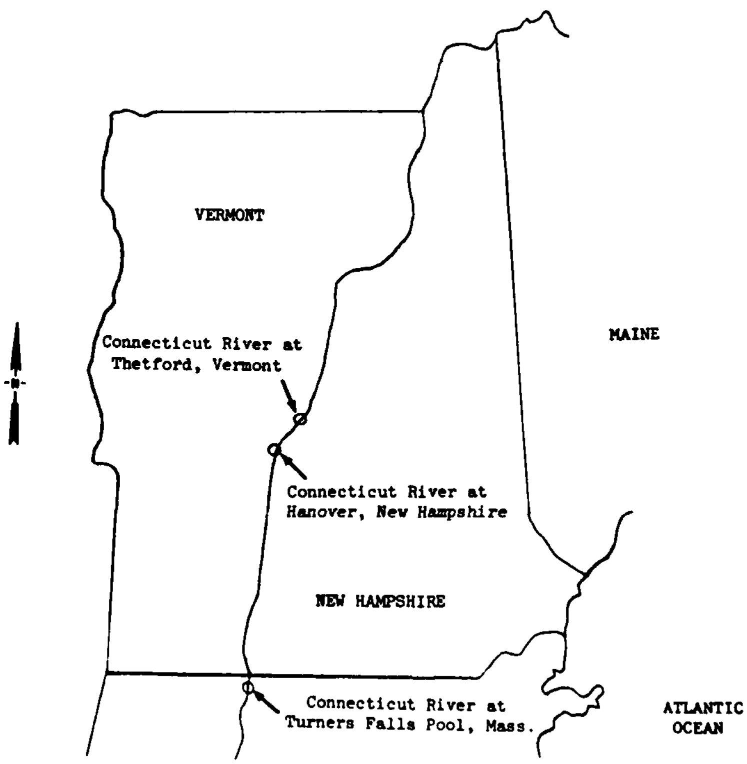
25-3 & 4 Project photographs

Connecticut River

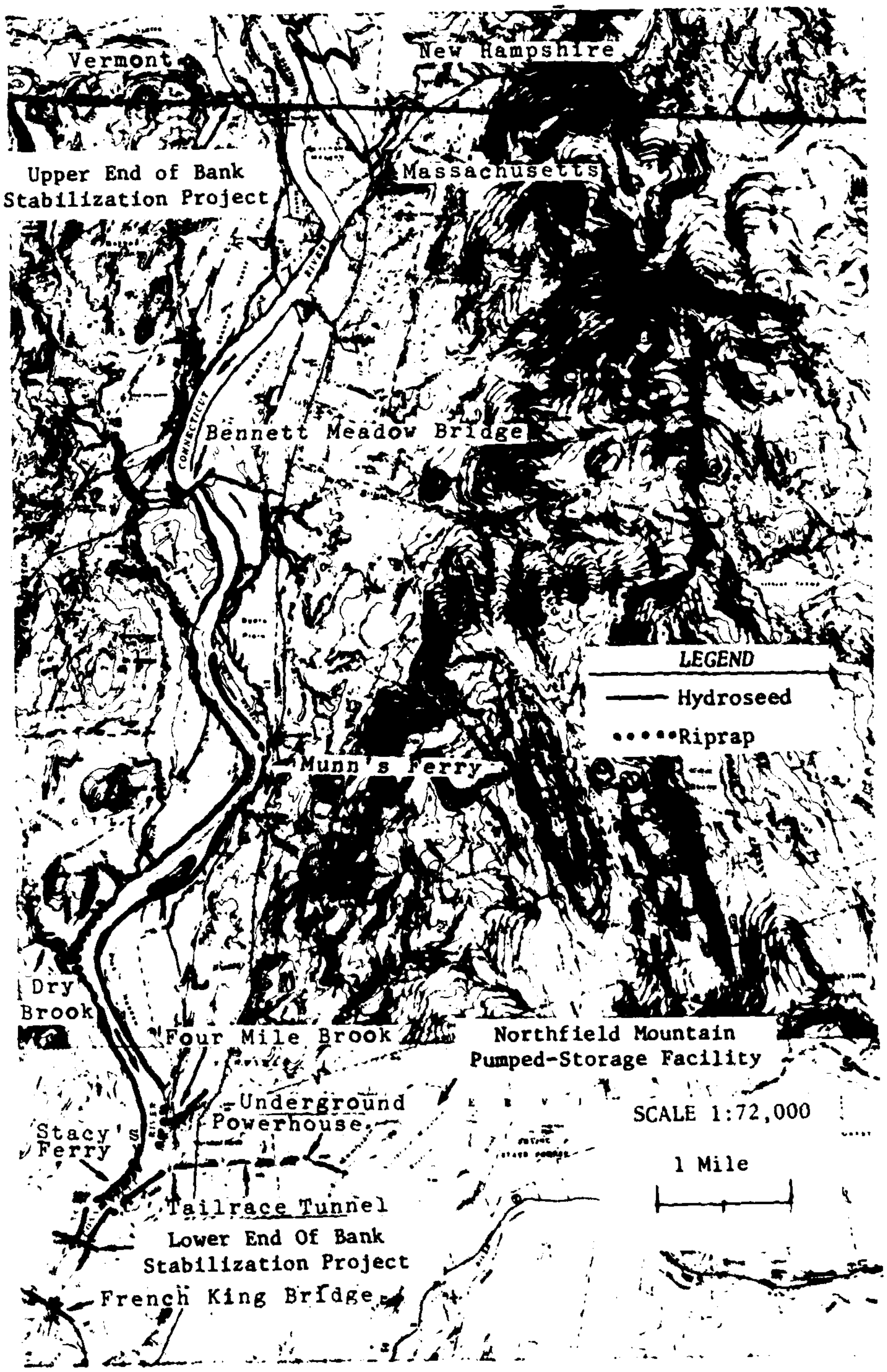
at Turner's Falls Pool, Mass.

In 1973, Northeast Utilities (NU) completed a pumped-storage generation facility and raised Turner's Falls Pool on the Connecticut River 5 ft. By 1975, a number of trees along the bank had toppled into the river as a result of erosion around their root masses, so NU removed all susceptible trees. After the trees were removed and the bank was exposed to sunlight, many volunteer grasses became established and thus enhanced the bank's resistance to erosion. NU hydroseeded 9 miles of banks in 1977 by barge. In those reaches where tree removal and hydroseeding did not effectively control erosion, the banks were riprapped. The stone riprap protection has remained intact since the project was completed; however, there have been no significantly high flows to sufficiently test it. The naturally steep banks that were hydroseeded show definite signs of erosion from overbank drainage, sloughing, and undercutting. The flatter slopes that were hydroseeded have been stabilized.

*Tree removal	20 miles	$350,000	1977
Hydroseeding	9 miles	$ 34,000	1977
Riprap	1.6 miles	$150,000	1978



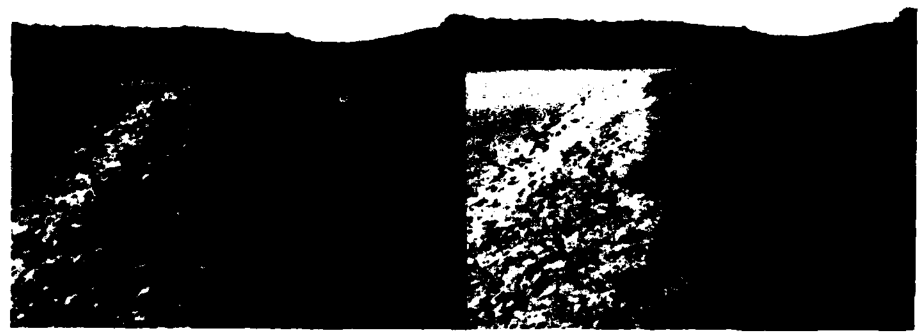
ITEM 25-1



Connecticut River at Turner's Falls Pool, Massachusetts (Source: USGS 1:24,000 topographic quadrangle maps for Northfield, Massachusetts, and Millers Falls, Massachusetts, 1976)

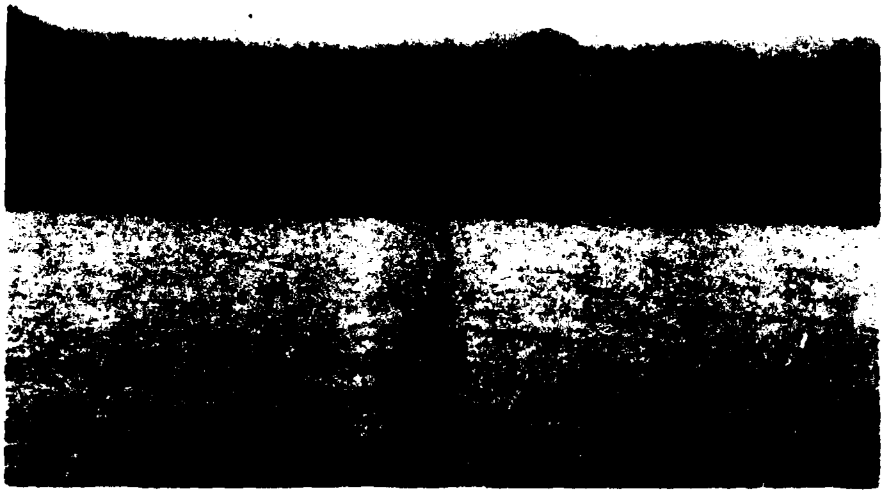
ITEM 25-2

Removal of trees by helicopter (photograph courtesy of Northeast Utilities)



Stereoscopic view of hydroseeded bank with riprap toe protection at Stacy's Ferry, Connecticut River at Turner's Falls Pool. The height from the water surface to the top of the revetment is approximately 4 ft, and to top bank, 12 ft. View is downstream (26 July 1978)

ITEM 25-3



Vegetation had become well established at the Stacy's Ferry site by winter of 1977-78, Connecticut River at Turner's Falls Pool. The height from the water surface to the top of the revetment is approximately 4 ft, and to top bank, 12 ft (photograph courtesy of Northeast Utilities)

Three years after placing riprap and hydroseeding at Stacy's Ferry, Connecticut River at Turner's Falls Pool (Oct 1980)

ITEM 25-4

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REPORT DOCUMENTATION PAGE

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Bank erosion
Bank protection

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

A preliminary study of streambank erosion control was conducted with the major emphasis on an extensive literature survey of known streambank protection methods. In conjunction with the survey, preliminary investigations were conducted to identify the mechanisms that contribute to streambank erosion and to evaluate the effectiveness of the most widely used streambank protection methods. The results of the literature survey and the two preliminary investigations are presented herein.

(Continued)

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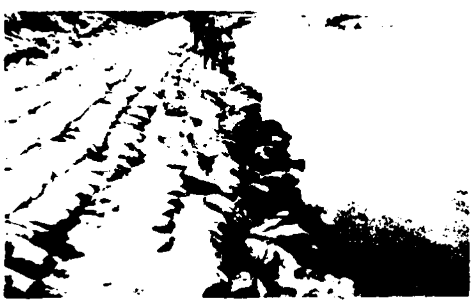
20. ABSTRACT (Continued).

The text of the "Streambank Erosion Control Evaluation and Demonstration Act of 1974" is presented in Appendix A. A list of commercial concerns that market streambank protection products is provided in Appendix B. Appendix C contains a glossary of streambank protection terminology. A detailed bibliography resulting from the literature survey is provided in Appendix D, and a listing of selected bibliographies related to streambank protection are provided in Appendix E.

THE STREAMBANK EROSION CONTROL EVALUATION AND DEMONSTRATION ACT OF 1974

SECTION 32, PUBLIC LAW 93-251

US Army Corps of Engineers



Sand-Cement Sack Revetment

Jack-Type Retards

Willow And Tire Revetment

Tire-Filled Cribs